Miscellaneous Paper HL-94-4
September 1994

US Army Corps of Engineers
Waterways Experiment Station

Flood Control Channels Research and Development Program

Streams Above the Line: Channel Morphology and Flood Control

Proceedings of the Corps of Engineers Workshop on Steep Streams, 27-29 October 1992, Seattle, Washington

Prepared for Headquarters, U.S. Army Corps of Engineers

PRINTED ON RECYCLED PAPER

Flood Control Channels Research and Development Program

Miscellaneous Paper HL-94-4
September 1994

Streams Above the Line: Channel Morphology and Flood Control

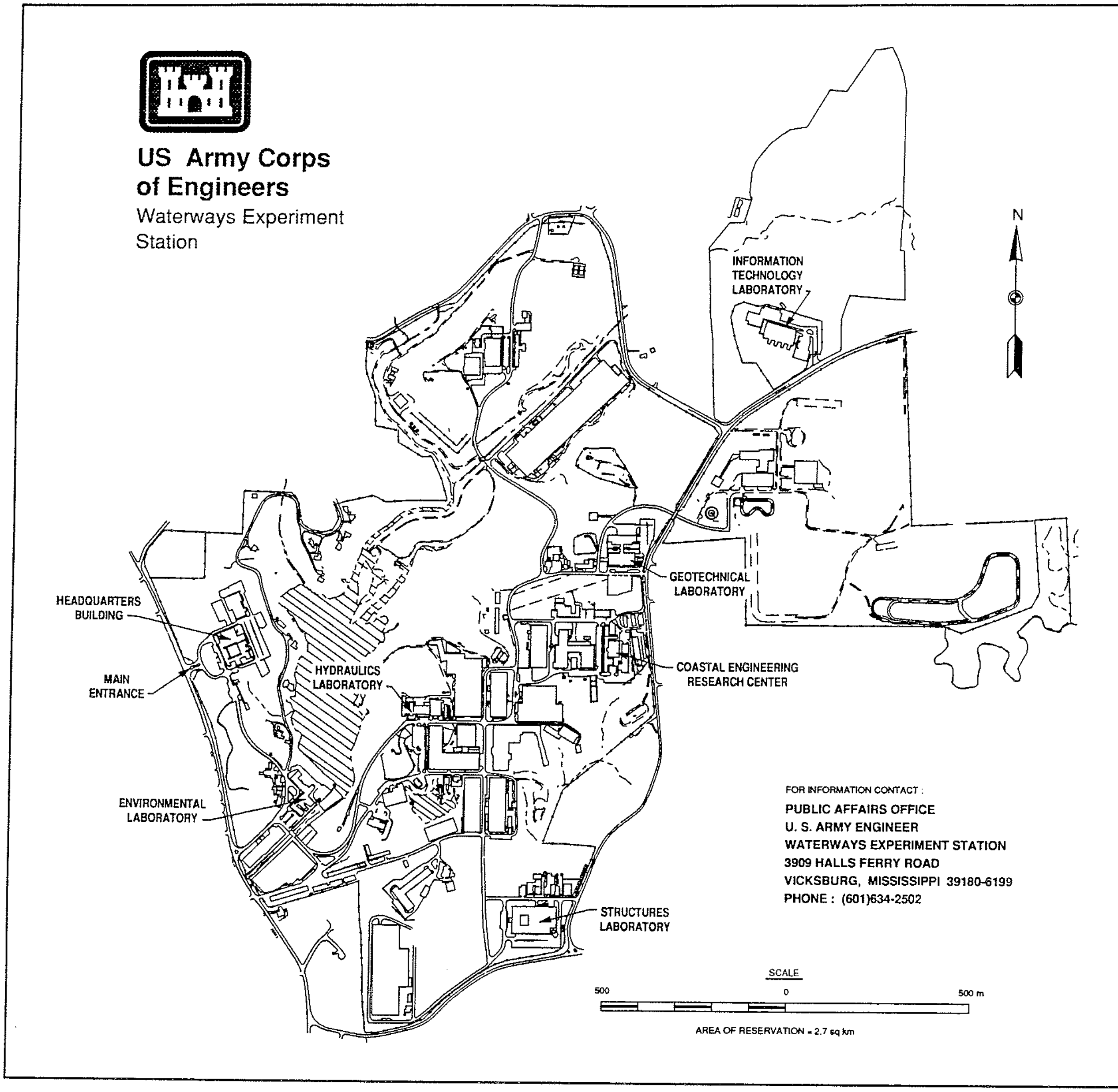
Proceedings of the Corps of Engineers Workshop on Steep Streams, 27-29 October 1992, Seattle, Washington

Final report

Approved for public release; distribution is unlimited

Prepared for U.S. Army Corps of Engineers
Washington, DC 20314-1000

Under Work Unit 32553



Waterways Experiment Station Cataloging-in-Publication Data

Corps of Engineers Workshop on Steep Streams (1992 : Seattle, Washington)
Streams above the line : channel morphology and flood control: proceedings of the Corps of Engineers Workshop on Steep Streams, 27-29 October 1992, Seattle, Washington / prepared for U.S. Army Corps of Engineers.
322 p. : ill. ; 28 cm. -- (Miscellaneous paper ; HL-94-4)
Includes bibliographic references.
1. River channels -- Congresses. 2. Flood control -- Congresses. 3. Geomorphology. I. United States. Army. Corps of Engineers. II. U.S. Army Engineer Waterways Experiment Station. III. Hydraulics Laboratory (U.S.) IV. Flood Control Channels Research and Development Program. V. Title. VI. Title: Proceedings of the Corps of Engineers Workshop on Steep Streams, 27-29 October 1992, Seattle Washington. VII. Series: Miscellaneous paper (U.S. Army Engineer Waterways Experiment Station) ; HL-94-4.
TA7 W34m no.HL-94-4

Contents

Preface

This report presents the proceedings of the U.S. Army Corps of Engineers (CE) Workshop on Steep Streams that was held in Seattle, WA, 27-29 October 1992. The workshop was sponsored by the Flood Control Channels Research Program, Work Unit No. 32553, "Gravel and Boulder Rivers," which is sponsored by Headquarters, U.S. Army Corps of Engineers (HQUSACE).

The organizational activities were carried out under the general supervision of Mr. William A. Thomas, Program Manager, Flood Control Channels Research Program, Waterways Division, Hydraulics Laboratory (HL), U.S. Army Engineer Waterways Experiment Station (WES). Mr. Frank Herrmann was Director, HL. The planning committee for the workshop consisted of Major Monte Pearson, Geotechnical Laboratory, WES; Mr. Thomas Munsey, HQUSACE; Mr. James Lencioni, U.S. Army Engineer District, Seattle; and Mr. John Oliver, U.S. Army Engineer Division, North Pacific. Mr. Lencioni made local arrangements. Dr. Bobby J. Brown, HL, WES, was responsible for coordinating the necessary activities leading to publication.

On the afternoon of the last day of the workshop, participants summarized the state of the art in, and discussed future directions for, engineering and scientific work on steep streams. They developed a list of unsolved problems with steep streams (Table 1) and ranked them according to the urgency of the problem. Although the necessary basic research is well under way, many knowledge gaps must be filled in before a standard steep channel design procedure is part of the technical literature. Knowledge of energy losses (hydraulic roughness) is a relative unknown for large materials as compared to particles up to sand sizes. A usable transport function for cobble size and above material has not been developed.

Total attendance at the workshop was 37; a list of participants and the Program are included. Persons not presenting papers were involved in discussion sessions and preparing summaries of discussions. During round-table discussion, all attendees provided input and voted on the most urgent unsolved problems with steep gradient streams.

At the time of publication of this report, Director of WES was Dr. Robert W. Whalin. Commander was COL Bruce K. Howard, EN.

Table 1
Flood Control Channel R&D

Most Urgent Unsolved Problems with Steep Gradient Streams	Priority		
	H	M	L
Hydraulic (grain, form and vegetative) roughness	21	0	1
Lack of proven method for sediment data collection and instrumentation	19	2	0
Debris basin design	14	5	0
Transport function(s) for large material	13	8	0
Training in fluvial geomorphology	13	7	3
Effects of vegetation on conventional (riprap) bank protection	13	4	4
Integrate the multi-objective approach into the COE planning process	12	3	6
Insight into the multi-dimensional aspects of sediment transport, including particle size, slope and longitudinal and lateral bed forms	11	9	1
Criterial for compound cross-section	11	8	1
Regime relationships	9	7	5
Illustrated manual of vegetative roughness values	9	7	5
Multi-laboratory approach to bio-engineering research	9	6	5
Debris flows	7	7	6
Low flow channel design with emphasis on environmental problems and solutions	6	12	3
High velocity lined channel design	6	6	8
Alluvial fan mechanics	5	9	7
Multiple-size analysis for large material gradation	5	8	7
Urban channel equilibrium dimensions, including dominant discharge	3	8	7
Fisheries in steep channels	2	4	15

List of Attendees

Townsend Barker
U.S. Army Engineer Division, New England
(617) 647-8220

Bill Branch
U.S. Army Engineer District, Portland
(503) 326-6470

Gary Brunner
USAE Hydrologic Engineering Center
(916) 756-1104

R. L. Burrow
U.S. Geological Survey
(907) 479-5645

Ed. F. Chacho Jr.
USAE Cold Regions Research Engineering Laboratory
(907) 353-5165

Jerri Comati
USAE Waterways Experiment Station
Duty Station: London, England
01-402-7331

Ronald R. Copeland
USAE Waterways Experiment Station
(601) 634-2623

Steve Custer
Montana State University
(406) 994-3331

David Hendrix
U.S. Army Engineer District, Nashville
(615) 736-5626

Eric Holand
U.S. Army Engineer District, Jacksonville
(904) 791-2241

James Lencioni
U.S. Army Engineer District, Seattle
(206) 764-3596

Steve Maynord
USAE Waterways Experiment Station
(601) 634-3284

Robert MacArthur
Resource Consultants and Engineers, Inc.
(916) 758-3611

Tom Munsey
Headquarters, U.S. Army Corps of Engineers, Hydraulics Branch
(202) 272-8504

Charles Neill
Northwest Hydraulic Consultants
(403) 436-5868

John G. Oliver
U.S. Army Engineer Division, North Pacific
(503) 326-3859

Monte L. Pearson
USAE Waterways Experiment Station
(601) 634-2118

James Pennaz
U.S. Army Engineer Division, Pacific Ocean
(803) 438-8599

Sam Powell
Headquarters, U.S. Army Corps of Engineers
(202) 272-8504

Nolan Raphelt
USAE Waterways Experiment Station
(601) 634-2634

Dick Reese
Northwest Hydraulic Consultants
(403) 436-5868

Ann Riley
Golden State Wildlife Federation
(510) 848-2211

Charles L. Rosenfeld
Oregon State University
(503) 737-1208

Mike Spoor
U.S. Army Engineer District, Huntington
(304) 529-5514

Scott Stonestreet
U.S. Army Engineer District, Los Angeles
(213) 894-5300

William A. (Tony) Thomas
USAE Waterways Experiment Station
(601) 634-2511

Brian Tracy
U.S. Army Engineer District, Los Angeles
(213) 894-5300

David Williams
WEST Consultants, Inc.
(206) 441-4212

David Mierzejewski (Scheduled to Attend)
U.S. Army Engineer District, Alaska
(907) 735-2504

Program for Steep Streams Workshop

October 27, 1992

Welcome	Monte L. Pearson
Keynote Address	Sam Powell
STEEP CHANNEL RESEARCH	
Practitioner's Overview	Charles Neill and Victor J. Galay
Why Steep Channels?	William A. (Tony) Thomas
Gravel Transport Measurement	Steve Custer
Radio Rocks Update	Ed. F. Chacho Jr., R. L. Burrow, and W. W. Emmett
Problems with Numerical Models of Gravel-Bed Rivers	Ronald R. Copeland and William A. (Tony) Thomas
STEEP CHANNEL DESIGN PRINCIPLES	
Roughness w/Bed Load Transport	Scott Stonestreet, Ronald R. Copeland, and Darla C. McVan
Rio Puerto Nuevo Sedimentation Study	Eric Holand and Brad Hall
Riprap Sizing for Steep Channels	Steve Maynord

October 28, 1992

FLOOD CONTROL ON ALLUVIAL FANS

Flamingo-Tropicana Alluvial Fan Flood Control Project	Brian Tracy

DEBRIS and MUD FLOWS

Debris Flows (Talk and Slide Presentation, No Paper)	Robert MacArthur
Mudflow Numerical Simulation	Gary Brunner
Field Trip	

October 29, 1992

SOCIO-ENVIRONMENTAL ASPECTS

Wildcat and San Pablo Cks., CA	Ann Riley
Restoring Atlantic Salmon in New England	Townsend Barker

LARGER STREAMS w/GRAVEL BEDS

White River/Mud Mountain Dam	James Lencioni
Floods in The Phillipines	Monte L. Pearson and John G. Oliver

GRAVEL STREAMS WITH GEOLOGIC CONTROLS

Yellow Creek, KY Flood Control Project	David Hendrix and Ronald R. Copeland
Iao Stream, HI Flood Control	James Pennaz
Incipient Motion Criteria Defining "Safe" Zones for Salmon Spinning Habitat in the Cedar River, Seattle, Washington (Paper not published or included in Proceedings)	David T. Williams

ROUNDTABLE DISCUSSION All

Design Problems w/Gravel Streams

New Gravel Stream Info Worthy of Corps-Wide Dissemination

Future Direction of Gravel Stream Research

END OF WORKSHOP

CORPS OF ENGINEERS WORKSHOP ON STEEP STREAMS
SEATTLE, OCT. 27-29, 1992.

SOME PHYSICAL CHARACTERISTICS OF STEEP STREAMS

by

Charles R. Neill and Victor J. Galay

Northwest Hydraulic Consultants
Edmonton and Vancouver, Canada
and Kent, Washington

ABSTRACT

Streams with gradients exceeding 25 ft/mile occur mainly as boulder torrents, braided channels, alluvial fan systems, or small gravel rivers. Diagrams are presented for steep channels linking gradient and Froude No. to bed-material size and depth or discharge, and certain inferences are drawn. Other topics discussed briefly include: relationships of planform type to gradient and bed-material size; debris flows; channel shifting; bed movement and transport; hydraulic roughness; and width-discharge relationships.

Several examples are presented of engineering studies involving steep alluvial streams with large-sized bed material. Key features and problems encountered in these studies are described and illustrated.

INTRODUCTION

The aim of the paper is to summarize some of the salient hydraulic and morphologic characteristics of steep streams from a practitioner's viewpoint, and to present some examples of problems in analyzing their behaviour. An extensive literature review has not been attempted.

Part 1 is in the nature of a brief overview. Part 2 presents six examples arising from the authors' experience over the last decade.

Collective references containing much information on steep streams include Hey et al (1982), Thorne et al (1987), Armanini and Di Silvio (1991), Beschta et al (1987), and IAHR (1991).

PART 1 - OVERVIEW

1.1 STREAM TYPES AND PHYSIOGRAPHIC SETTINGS

Steep streams, defined here as those having gradients exceeding 25 ft/mile or 0.005, appear to occur mainly in four forms: (1) boulder torrents including so-called debris flows or torrents; (2) braided channel systems; (3) alluvial fan and bajada systems including ephemeral streams; and (4) relatively small gravel rivers with "wandering" or meandering types of planform. Larger gravel rivers generally have gradients flatter than 0.005.

With respect to geomorphic processes, these various forms of steep stream have little in common except steepness and relatively coarse bed material. Boulder torrents occur mainly in mountain ravines and valleys and generally represent an erosional environment. Braided channel systems usually occur in coarse alluvial valley fills, often of glacial outwash: it is often unclear whether they are erosional nor depositional. Streams on alluvial fan and bajada (piedmont fan) systems are usually but not always depositional. Small gravel rivers often are neither obviously erosional or depositional.

Although in very general terms erosional environments tend to occur nearer the source and depositional environments nearer the mouth of a stream system, there are many exceptions due to geological and tectonic features. Actively depositing alluvial fans and bajadas often occur where the stream emerges from mountains near the upper end of the system, but they may also occur near the mouth of the stream if it emerges on to a coastal plain or alluvial valley. Erosional reaches may be found near the mouth if the stream cuts through a coastal mountain or hill range. Therefore the various forms of steep stream are not necessarily restricted to particular segments (from source to mouth) of a specific stream system.

Steep streams in glaciated regions are often located on thick coarse-grained deposits of glaciofluvial origin (outwash). Short steep gravel rivers with relatively little change in gradient are characteristic of narrow plains between high mountain ranges and the coast, as in New Zealand and Chile. On the other hand, stream systems with their source in far-inland mountain chains tend to have long upward-concave profiles where the bed material becomes gradually finer and the gradient flatter in the downstream direction and the final phase is a meandering sand stream - as frequently occurs on the east side of the North American Cordillera.

1.2 GRADIENTS AND PROFILES IN RELATION TO BED MATERIALS AND FLOW PARAMETERS

A considerable proportion of alluvial streams with gradients exceeding 0.005, tend to exhibit near-critical or supercritical flow under most flow conditions. Figure 1 represents a relationship for channels linking gradient, median bed-material grainsize, depth of flow under "just-mobile" conditions and Froude Number. It is based on the assumptions that (1) the Shields Number for just-mobile conditions is 0.045, (2) the dry relative density of the bed material is 2.6, and (3) the absolute roughness k is 3 times the median grainsize. These assumptions are not universally valid but are useful for illustrative purposes. Accepting them for present purposes, the following inferences can be drawn from the diagram:

- Gradients exceeding 0.005 imply median grainsizes exceeding 50 mm, unless the "just-mobile" depth is less than 3 ft.

- Gradients exceeding 0.005 correspond to Froude Numbers exceeding 0.75.

- Gradients exceeding 0.012 or so correspond to supercritical flow.

- Alluvial streams with median grainsizes exceeding 300 m, in the boulder torrent category, will mostly have gradients exceeding 0.01 and will exhibit near-critical or supercritical flow.

Gradients exceeding 0.005 can exist with median bed-material sizes finer than 50 mm if the just-mobile condition occurs at small enough depths and discharges. This is the case on some alluvial fan and bajada systems, especially if the stream is ephemeral. (In streams of laboratory scale, gradients exceeding 0.005 can be just-mobile with grainsizes as small as 5 to 10 mm.)

Figure 2 shows another form of diagram for coarse alluvial streams that links gradient, median bed-material grainsize, Froude Number, and "just mobile" discharge (the last replacing the depth variable in Figure 1). It is based on the same assumptions as Figure 1, with additional assumption that channel width (W) is related to the just-mobile discharge (Q) by the equation $W = CQ^{0.5}$, where C is 1.8 in fps units (3.26 metric).

Again, these assumptions are not universally valid - they are acceptable for many single-channel gravel streams and possibly for individual channels of a braided system, but are incorrect for a multi-channel or braided system taken as a unit. For single-channel alluvial systems, the following inferences can be drawn from Figure 2:

- As the scale of the stream system increases, the same gradient corresponds to larger grainsizes, or the same grainsize corresponds to flatter gradients.

- Gradients exceeding 0.005 imply median grainsizes exceeding 50 mm unless the just-mobile discharge is less than 700 cfs.

- Boulder torrents (median grainsize 300 mm) will exhibit supercritical flow unless the just-mobile discharge exceeds 30,000 cfs.

Figures 1 and 2 imply single-channel alluvial streams with even bed profiles where hydraulic resistance is mainly due to grain roughness. But many steep streams are not fully or even partially alluvial, and hydraulic resistance may derive from many sources other than grain roughness, eg. rock outcrops, abrupt cross-sectional irregularities, lag deposits of large boulders, sharp bends, and bank or overbank vegetation. In fact, steep mountain streams seldom have an even bed profile, but instead exhibit a series of steps and pools, as described for example by Whittaker (1987). In these cases overall supercritical flow may appear only at very high discharges when the steps and pools are drowned out.

The longitudinal profile stability of a particular segment of stream - whether degrading, aggrading or vertically stable - is very difficult to determine except where there are long-term stage-discharge records. Although general inferences about long-term trends on a geologic timescale may be made from the type of geomorphic environment, these may be of little value for engineering purposes. For instance, it can be argued that a steep braided river segment that changes fairly abruptly to a much flatter segment must be advancing over the latter and therefore aggrading; however the process may be so slow on a human timescale that the aggradation is virtually undetectable. Also, changes due to tectonic processes, seismic events or human interference may be large enough to mask or overwhelm such slow processes.

1.3 DEBRIS FLOWS AND TORRENTS

Debris flows or debris torrents (both terms are used) represent a special type of extremely steep stream in mountainous terrain. Debris flows differ from ordinary flood flows with high sediment transport, in that there is a surging flow involving a high-density fluidized mass of boulders, sand and gravel, mud, logs and other organic debris, with high destructive power. While debris flows can occur in strictly natural conditions, there is little doubt that many instances have resulted from catchment disturbance by logging, road-building and other activities. In those cases the causative mechanism may be de-stabilization of the ground surface, or increase in flood peak runoff, or diversion and concentration of flow into certain channels. Debris flows are usually associated with relatively rare rainstorm or flood events.

Streams liable to debris flows are extremely steep in relation to the lower gradient limit (0.005) adopted herein. Observations indicate that a debris flow can only be initiated on gradients exceeding about 15 degrees (0.26). Such gradients occur only in relatively small streams in steep mountain ranges. Once initiated, however, a debris flow may not stop until it reaches a point where the gradient is reduced to 5 degrees (0.09).

1.4 CHANNEL PLANFORMS IN RELATION TO GRADIENT

Steep streams exhibit a variety of planform types besides the classical "meandering, braided and straight" of Leopold and Wolman (1957). Regular well-developed meanders of more or less constant wavelength and amplitude - common in flat-gradient streams - are rare, if they exist at all. Steep streams in alluvial valleys may have an irregular type of meander pattern with frequent midstream bars and islands that is sometimes called "wandering"; if most of the stream length exhibits more than one channel, but is not fully braided, it may be called "anastomosing". Boulder torrents frequently have long almost-straight lengths and irregular bends that are controlled largely by the surrounding topography. Streams on fans and bajadas tend to show a distributary pattern something like those on deltas, although only one or two channels may be active at a given period.

Various efforts have been made to chart the relationship of planform type to gradient and other factors. Three examples are as follows:

- Figure 3 after Leopold, Wolman and Miller (1964) shows a single line on a slope-discharge diagram, supposed to discriminate between meandering and braided patterns for gravel rivers. Most of the relevant data are for gradients of less than 0.005, not "steep" as defined herein.

- Figure 4 by Kellerhals and Church (1989) shows an overlap zone between an upper limit for single-thread channels and a lower limit for braided channels. An intermediate category of "wandering" channels - neither single-thread nor fully braided - is recognized. This diagram shows more data points in the "steep" range.

- Figure 5 is an earlier diagram by Ferguson (1984), which shows mostly the same data set as Figure 4 but includes grainsize as an additional variable. The effect of grainsize appears to be small empirically on the basis of the data set, but much more significant theoretically. The theoretical threshold lines for braided channels are based on work by Parker (1979).

Figures 3, 4 and 5 give widely different indications on the relationship of channel planform type to gradient. According to Figure 3, all steep streams ($S > 0.005$) would be braided unless the bankfull discharge were less than about 10 m^3/s or 350 cfs. According to Figure 4, steep streams could have single channels at discharges up to about 300 m^3/s or 10,000 cfs. According to the theoretical lines of Figure 5, steep streams can be non-braided at almost any scale as long as the bed material is coarse enough: for example with a median grainsize of 256 mm or 10 inches, gradients in the range of 0.01 to 0.1 would be required for braiding. Boulder torrents can in fact be non-braided on very steep gradients.

1.5 LATERAL PROCESSES IN STEEP STREAMS

Lateral erosion and channel shift processes in steep streams tend to be irregular and difficult to predict in comparison to those in plains rivers. The most troublesome type in this respect are streams on actively aggrading fans and bajadas, which tend to build up their beds and become "perched" with respect to the adjacent fan surface. These streams are liable to avulsions - sudden breakouts from the prevailing channel to form a new channel - during high flood events as the old channel blocks with bed material. Because of the way the channel system fans out in the downstream direction, the main channel location towards the lower end of the system may shift dramatically in such an event, leaving riparian facilities high and dry and destroying developments in the way of the new route.

The type of regular meander pattern migration that is often seen in plains rivers, and that makes prediction of future channel locations relatively easy on the basis of past mapping or airphotography, is seldom seen in steep streams, although individual well-developed bends may migrate in a similar fashion.

Log jams often have a strong influence on the lateral processes of steep streams in forested regions, blocking secondary channels and inducing sudden random shifts.

1.6 BED MOVEMENT AND TRANSPORT

In steep non-braided streams with coarse bed material, significant bed transport tends to occur only at relatively high discharges - in the order of the mean annual flood. Over a period of years, occasional large floods may be responsible for most of the total transport. Many such streams exhibit bed armoring, between transport episodes the bed is covered selectively with a layer of the larger grains in the bed-sediment mixture and is relatively resistant to movement. In the case of lake or reservoir outlet channels where there is practically no supply of bed material, bed movement may be restricted to rare floods. References on armoring process include Sutherland (1987) and Andrews and Parker (1987).

In steep streams of the braided or alluvial fan type, transport is likely to be much more frequent and armoring may not occur.

The reliability of the more popular sediment transport formulas and procedures is relatively untested for steep streams in comparison to those of flatter gradients. Some of the earlier formulations, for example Meyer-Peter and Muller (1948), may be as good as any because they were derived from data sets that included quite steep gradients and coarse materials. Bathurst et al (1987) tested a number of procedures against data for steep flumes and rivers, and favoured an equation of even older origin by Schoklitsch. They pointed out, however, that with gradients exceeding 0.01, bed material supply is often limited and all formulas tend to over-predict. This comment is particularly applicable to boulder torrents in erosional settings, where bed material may be supplied to the stream only from occasional episodes like landslides and tributary debris flows. Cases of this type are analyzed by Whittaker (1987).

1.7 CHANNEL ROUGHNESS AND HYDRAULICS

As is well known, total hydraulic resistance in natural streams can arise from a variety of sources including: grain roughness of surficial bed material; bed-form roughness from ripples, dunes and bars; rock outcrops; debris or ice accumulations; abrupt irregularities in plan and cross-section including local scour holes; channel divisions and confluences; surface irregularities in near-critical and supercritical flow; and vegetation. In many steep streams with coarse bed material, grain roughness is a dominant source - in contrast to plains rivers with finer bed material, where bed-form roughness is often dominant. There is an extensive literature on the topic: examples particularly applicable to steep streams include Simons et al (1979), Bathurst (1982, 1985) and Aguirre-Pe and Fuentes (1990).

It is still common to characterize stream roughness by the Manning coefficient, but many steep streams have such high ratios of roughness height to depth that the Manning formula is not correctly applicable. Manning roughness values determined at different stages of flow may vary over a considerable range, partly because of the incorrect mathematical form of the equation at high relative roughnesses, and partly because additional sources of roughness - such as bank vegetation - may only come into play physically above a certain stage. It can therefore be unreliable to determine flow profiles for a range of flows on the basis of uncalibrated coefficients, or even of coefficients calibrated at one flow condition. Yet numerous engineering studies continue to exhibit uncritical acceptance of non-uniform flow computations that do not consider variability of roughness values, nor alteration of cross-sections by sediment transport and scour.

A number of flow formulas for estimating velocities in steep streams, independent of Manning roughness, are discussed by Thorne and Zevenbergen (1985). A comparison with data is somewhat inconclusive. An empirical formula by Lacey (1933-34) that uses only depth and slope as input, and that the present author has often used for rough estimates in a range of environments, was found to be inadequate for mountain rivers.

1.8 CHANNEL WIDTH IN RELATION TO DISCHARGE

There is a general trend for the width of stream channels to be approximately proportional to the square root of representative discharge, over a wide range of discharges. Figure 6, based on Kellerhals and Church (1989), shows this trend over an extreme discharge range.

There is considerable evidence that, the most appropriate discharge for width correlations is the bankfull or equivalent. In terms of flood frequencies, this often corresponds to a median annual event, but in streams with steep frequency curves it may be closer to a 5-year or even a 10-year event.

In the general relationship $W = CQ^{0.5}$, a value of C in the vicinity of 2.0 (fps units) or 3.6 (metric) often seems to give a reasonable correlation for single-channel coarse-bed streams. In multiple-channel and braided streams, a similar value is applicable to individual channels of the system. One application of this relationship is to estimate bankfull discharge from airphotos. In selecting a width for input to the formula, attention should be given to relatively straight lengths of even width, neglecting migrating bends or with wide gravel bars.

Boulder torrents often seem to yield considerably smaller values of C, in the range of 1 to 1.5, but the author is not aware of systematic data.

PART 2 - EXAMPLES

The following six examples of steep streams are based on consulting studies by the authors. For each case, a brief description is given of the key stream characteristics and the nature of the engineering problem.

2.1 BRAIDED BOULDER RIVER - BURIED CONDUIT CROSSINGS

The Rio Maule downstream of Colbun Dam in south-central Chile is a braided river with a slope of 0.0065 (43 ft/mile). Bed material is characterized by a D_{50} size of 75 mm, a D_{90} size of 230 mm, and occasional boulders up to 700 mm. The overall width of the braided system averages around 900 m. Most cross-sections show three or four well-defined low-flow channels with widths ranging from about 50 to 150 m, separated by gravel and boulder bars. The flood flow regime is characterized by 2-, 10-, and 100-year floods of 2300, 3800 and 6200 m^3/s. The river flows over an irrigated sloping plain or bajada formed of alluvial outwash from the Andes mountain range. Bankfull discharge is in the vicinity of 3000 m^3/s.

Construction of a high dam at the outlet from the mountains was expected to cut off the bed material supply to the river. Two large irrigation conduits had to be constructed under the river, one about 3 km downstream of the dam and the other about 23 km downstream. Estimates were required of potential scour and degradation of the river bed over the buried conduits.

The upper conduit crossing was clearly within the zone that could be expected to degrade vertically within a foreseeable time of dam completion. Estimates were made both by numerical modelling and by empirical methods. Uncertainties arose over the effect of armoring and the response of the channel planform to removal of the bed material supply. It was generally accepted that armoring would occur and that the planform would modify to produce one or at the most two channels. Results indicated, for a 50-year period, a maximum degradation of about 5 m immediately below the dam, tailing out to zero at about 5 km downstream (Figure 7).

The lower crossing was too far downstream to be affected by profile degradation within a reasonable period of time. For an unconstricted crossing, estimates were made of potential scour in major floods. These were relatively small - in the order of 2 m below the deepest points surveyed in low water conditions. Rather than crossing the entire braided width of 900 m with deep burial, it was suggested to reduce the length of deep burial to about 650 m and provide training dikes to confine the river accordingly. For a construction method involving staged cofferdamming of the river, it was recommended that a clear width of 300 m be left for flood flows at all times.

2.2 WANDERING BOULDER RIVER - CHANNELIZATION

The Rio Segre at the town of Seo d'Urgell in northeast Spain has a slope of approximately 0.008. The bed material is characterized by a D_{50} size of 150 mm, a D_{90} size of 400 mm, and occasional boulders up to about 1 m. The river emerges from the edge of the Pyrenees mountains just above the town, and then flows through an irregular alluvial floodplain that borders the town and has a width varying from about 100 to 500 m. In its natural state the planform exhibited a mainly single channel forming a series of irregular meanders of relatively low sinuosity. The flood flow regime is characterized by 2-, 10- and 100-year peak flows of 150, 430 and 810 m^3/s. The average channel width was about 30 m.

A large flood in 1982 caused extensive changes to the planform of the river and substantial damage to riparian and floodplain facilities. The river shifted extensively over much of the length, developing secondary channels, gravel and boulder bars, and abrupt bends subject to rapid erosion. It was decided to revive a proposal first mooted a century before, for channelizing the river through the town. The channelization was completed in 1989 and incorporated intake facilities for a kayak race course built for the 1992 Olympic Games.

The channelized river has a trapezoidal cross-section of 70 m bed width and 2.5 m depth, designed to accommodate a 50-year flood of 700 m^3/s. It has a sinuous alignment with long bends of 500 m radius, following as closely as possible the natural course of the river (Figure 8). The banks are stabilized with riprap but the bed is of natural river-bed material. The stabilized banks have 1 m freeboard and rise about 1.5 m above the floodplain. The design mean velocity was approximately 4 m/s and the Froude Number about 0.85. Three classes of riprap were used for bank protection: heavy, nominally 800 mm, on the outsides of bends; medium, nominally 500 mm, on straight lengths between bends; and light, nominally 300 mm, on the insides of bends.

It was recognized that the new channel was considerably oversized for ordinary flows and annual floods, and that maintenance would probably be required because of continued supply of bed material from upstream. Photographs taken about two years after completion indicate deposition of sediment, growth of vegetation and development of an irregular inner channel inside the stabilized banks. In retrospect, a compound cross-section might have been preferable.

2.3 SPILLWAY OUTFLOW CHANNEL "X" - PMF DEGRADATION POTENTIAL

Channel "X" is a boulder torrent type of channel below a dam spillway and plunge pool that have been in place for about 65 years. The slope averages about 0.015, and the bed material consists of randomly scattered large boulders, some up to several metres in size, overlying coarse gravel and cobbles. The natural flood flow regime was characterized by 2-, 10- and 100-year floods of around 230, 350 and 490 m^3/s, but since dam construction only occasional outflows have occurred, with a maximum value of about 450 m^3/s. Typical annual releases are believed to be in the order of 50 m^3/s. In response to the reduced flows, the gross width has narrowed from perhaps 50 m to about 20 m by encroachment of dense forest vegetation on formerly clear overbank areas. The original width is bordered by steep valley slopes with dense vegetation.

The dam facilities were to be upgraded to provide for a Probable Maximum Flood. The PMF hydrograph has a peak of 1400 m^3/s and exceeds the original dominant or channel-forming discharge - estimated at 300 m^3/s - for over 3 days. In connection with design of a new stilling basin, estimates were required of potential degradation of the outflow channel under a PMF. Available information suggested that the large-sized bed material was basically a surface layer overlying sand and gravel. Hydraulic computations based on (1) the existing width, and (2) resumption of the original width indicated PMF velocities from 6 to 7 m/s and depths of 5.5 to 6 m.

It was estimated that the bulk of the surficial bed material - up to about 0.6 m in size - could be set in motion by flows of around 800 m^3/s, which persist for 1.5 days during a PMF. The largest boulders of 2 m size or greater were estimated to be only marginally unstable at the PMF peak. These boulders are probably a lag deposit resulting from entrenchment of the original stream into glaciolacustrine deposits that contain ice-rafted rock fragments.

It was recognized that reliable estimation of degradation potential was difficult for a number of reasons, including (1) lack of sufficient subsurface information, (2) unreliability of transport functions for such large material, (3) the apparent layered nature of the bed and the random distribution of large boulders, and (5) the uncertain role of lateral erosion in a PMF event. Rather than relying on dubious sediment transport computations, upper and lower limits on degradation at the head of the channel were based on available geological and field information, the observed distribution of large boulders, the plotted channel profile, the above-mentioned estimates of hydraulic stability, and expected minimum slopes if the surface layer were removed. It was estimated that PMF degradation might extend over a channel length of 200 to 300 m and involve 2 to 4 m lowering of tailwater at the stilling basin (Figure 9). Consideration was given to installation of a rock sill across the channel to control degradation.

2.4 SPILLWAY OUTFLOW CHANNEL "Y" - PMF DEGRADATION POTENTIAL

The upstream part of Channel "Y" is an artificial cut below a spillway, that joins a previous natural stream about 400 m downstream. Over the first 600 m, the slope increases from practically zero to about 0.03 (160 ft/mile) and then continues around that value. The bed surface layer consists mostly of cobbles and boulders, typically 300 to 400 mm in size and about 0.5 m thick, overlying sand and gravel with some sandy clay. The cut channel has a bed width of 12 to 15 m and steep wooded side slopes, while the natural channel farther downstream has a wooded floodplain around 40 m wide. The cut channel was built almost horizontal with a steep drop at its downstream end, but has degraded up to 8 m to attain its present profile. Over a period of about 40 years since dam construction there have been only occasional outflows of up to 100 m^3/s or so.

As in the case of Channel "X", degradation estimates were required for a PMF hydrograph, in this case with a peak of 600 m^3/s and a timebase of about 48 hours. Hydraulic computations based on a fixed bed indicated that the flow would be mostly supercritical with velocities of around 6 m/s. However the potential erosional response in much greater than that of Channel "X" because of the lack of large boulders to control degradation. Estimates of potential degradation were based on considering initiation of movement for the surface layer, stable slopes for the underlying material, and transport rates of the underlying material, dividing the PMF hydrograph into a series of steps. The volume removed after a given time was computed on the basis of the transport capacity of the existing channel at the downstream end of the degradation wedge, on the principle of sediment balance (Neill 1987). The volume was then distributed in the form of a curvilinear wedge (Figure 10) with the degraded profile tangential to a stable slope at its upstream end and to the existing slope at its downstream end.

As in the case of Channel "X", numerous uncertainties were recognized in the available information and the method of attack. Sensitivity was tested by assuming first no widening of the existing channel, and then widening to several times the existing width. Various scenarios were adopted for removal of the surface layer, and different sediment transport functions were used. (Because of the very high stream power, computed transport of the underlying material is quite insensitive to grainsize.) Estimates were made both with and without a downstream control point.

It was concluded that uncontrolled degradation at the head of the outflow channel could amount to 6 m with widening assumed, and up to 10 m or more with no widening. The length of the degradation wedge was about 600 m. Estimates were then made for degradation below a proposed tailwater control structure to be located 100 to 200 m downstream of the spillway. As might be expected, similar results were obtained. Consideration was therefore given to further subsurface investigations and to the feasibility of scale model studies.

2.5 BOULDER TORRENT - HYDROELECTRIC INTAKE

The Ok Menga is a torrential mountain stream in rainforest in western Papua New Guinea. Slopes range locally from 0.01 to 0.08 and flow is generally near-critical or tumbling. The bed and banks consist mostly of material exceeding 100 mm and contain boulders up to 6 m in size, in places as accumulations and falls. The stream is actively degrading into unstable colluvial deposits which in places form near-vertical cliffs. Recorded flows in a short record period have ranged from under 5 to over 900 m^3/s, and flows can vary daily over a wide range. The channel varies in width from about 20 to 50 m and depths range from about 1 m at low flow to 10 m in a 100-year flood, estimated at 1700 m^3/s. The 100-year discharge intensity is up to 40 m^3/s per m width. During floods the stream transports considerable quantities of coarse sand and gravel - probably in saltation or suspension - as well as cobbles and small boulders as bed load.

To supply a hydroelectric plant servicing a mining operation, a diversion weir and sediment-excluding intake were to be designed, leading to a drop shaft and long tunnel. The weir and intake were designed with the aid of a hydraulic model, constructed and calibrated on the basis of field observations. The model scale was 1:45 undistorted, with a partially mobile bed. The model covered a channel length of 500 m. Large boulders and other features were reproduced as closely as possible using helicopter photography.

The final weir design involved cast-in-place concrete blocks several metres in dimensions in a stepped pattern, with a downstream riprap apron of 2 m stones. The concrete intake structure was provided with arrangements for excluding bed load and for settling coarse suspended and saltating load and returning it to the river downstream. Early operating experience revealed problems with accumulation of organic trash and finer sediment, that had been somewhat overlooked in the model studies, but on the whole the design appears to have overcome the worst severe problems of an extremely difficult site.

More details on this installation are available in an article by McCreath et al (1990).

2.6 STEEP STREAM WITH ABRUPT CHANGES IN SLOPE AND WIDTH

The Chilliwack River in the Fraser Valley east of Vancouver, British Columbia has alternating steeper and flatter reaches with slopes of around 0.0125 and 0.007 (Figure 12). The middle steep reach, basically erosional in character, crosses glacial deposits of clay and silt overlying gravel outwash, and is lined with boulders up to 1.5 m in size as well as with bank riprap. The boulders are believed to represent lag deposits, possibly accumulated in a glacial-lake outflow channel. The flatter reaches upstream and downstream are basically depositional in character and carry large bed loads of coarse gravel, partly supplied episodically from landslides upstream. The flood flow regime is characterized by 2-, 10-, and 100-year flood peaks in the order of 500, 1000 and 2000 m^3/s.

The steeper and flatter reaches exhibit marked differences in planform and cross-section. The steep reach has a single channel approximately 50 m wide. The flatter reaches have multiple channels totalling about 100 m within an overall braided system about 400 m wide. Bankfull discharge in the lower braided reach was naturally around 500 m^3/s but has been substantially increased by a flood control dike and road embankment.

Because of urbanization of the area, land developers are trying to encroach on the active floodplain and to promote river training works and gravel dredging programs in support of such developments. These pressures are being resisted by sport fishermen and environmentalists. Planning decisions will depend on multidisciplinary reviews and on cost estimates for river modification, which depend in part on realistic estimates of net bed-material accumulation in the reach. It is known that the gravel bed material disappears a few kilometres downstream, which indicates that it must be accumulating in the braided reach. Sediment transport calculations have proved difficult to verify, but attempts are being made to compare cross-sections before and after floods and to measure bed load at selected locations.

REFERENCES

Aguirre-Pe, J. and Fuentes, R. 1990. Resistance to flow in steep rough streams. ASCE Jour. Hydr. Engin. v.116 no.11 p.1374.

Andrews, E.D and Parker, G. 1987. Formation of a coarse surface layer as the response to gravel mobility. In Thorne et al 1987 (see below).

Armanini, A. and Di Silvio, G. (editors) 1991. Fluvial hydraulics of mountain regions. Lecture Notes in Earth Sciences no. 37, Springer-Verlag.

Bathurst, J.C. 1982. Theoretical aspects of flow resistance. In Hey et al 1982 (see below).

Bathurst, J.C. 1985. Flow resistance estimation in mountain rivers. ASCE Jour. Hydr. Engin. v.111 no 4 p.625.

Bathurst, J.C., Graf, W.H and Cao, H.H. Bed load discharge equations for steep mountain rivers. In Thorne et al 1987 (see below).

Beschta, R.L., Blinn, T., Grant, G.E., Ice, G.G. and Swanson, F.J. 1987. Erosion and sedimentation in the Pacific Rim. Intern. Assoc. of Hydrol. Sci. Publ. no. 165.

Ferguson, R.I. 1984. The threshold between meandering and braiding. In conf. proc. "Channels and Channel Control Structures", ed. K.V.H. Smith, Springer-Verlag.

Hey, R.D., Bathurst, J.C. and Thorne, C.R. 1982 (editors). Gravel-bed rivers: fluvial processes, engineering and management. John Wiley and Sons.

IAHR (Intern. Assoc. for Hydraulic Research) 1991. Study of streams and watersheds of high hydraulic irregularities. Vol. A, X

Kellerhals, R. and Church M. 1989. The morphology of large rivers: characterization and management. In Proc. Intern. Large Rivers Symp., Canadian Special Publication of Fisheries and Aquatic Sciences, Ottawa.

Lacey, G. 1933-34. Uniform flow in alluvial rivers and canals. Min. Proc. Inst. Civ. Engrs. vol.237 p.421.

Leopold, L.B. and Wolman, M.G. 1957. River channel patterns; braided, meandering and straight. USGS Prof. Paper 282-B.

Leopold, L.B., Wolman, M.G. and Miller, J.P. Fluvial processes in geomorphology. W.H. Freeman & Co.

Meyer-Peter,E. and Muller, R. 1948. Formulas for bedload transport. Intern. Assoc. Hydr. Research, 2nd Congress, Stockholm.

McCreath, P.S., Neill, C.R., Sawatsky, L.F. and Mannerstrom, M.C. 1990. River intake works for a hydroelectric plant in Papua New Guinea. Canadian Jour. Civ. Engineering vol.17 p.578.

Neill, C.R. 1987. Sediment balance considerations linking long-term transport and channel processes. In Thorne et al 1987 (see below).

Parker, G. 1979. Hydraulic geometry of active gravel rivers. ASCE Jour. Hydr. Div. vol. 105 p. 1185.

Simons, D.B., Al-Shaikh-Ali, K.S. and Li, R-M. 1979. Flow resistance in cobble and boulder riverbeds. ASCE Jour. Hydr. Div. vol. 105 p. 477.

Sutherland, A.J. 1987. Static armour layers by selective erosion. In Thorne et al 1987 (see below).

Thorne, C.R. and Zevenbergen, L.W. 1985. Estimating mean velocity in mountain rivers. ASCE Jour. Hydr. Engin. vol.111 no.4 p.612.

Whittaker, J.G. 1987. Sediment transport in step-pool streams. In Thorne et al 1987 (see above).

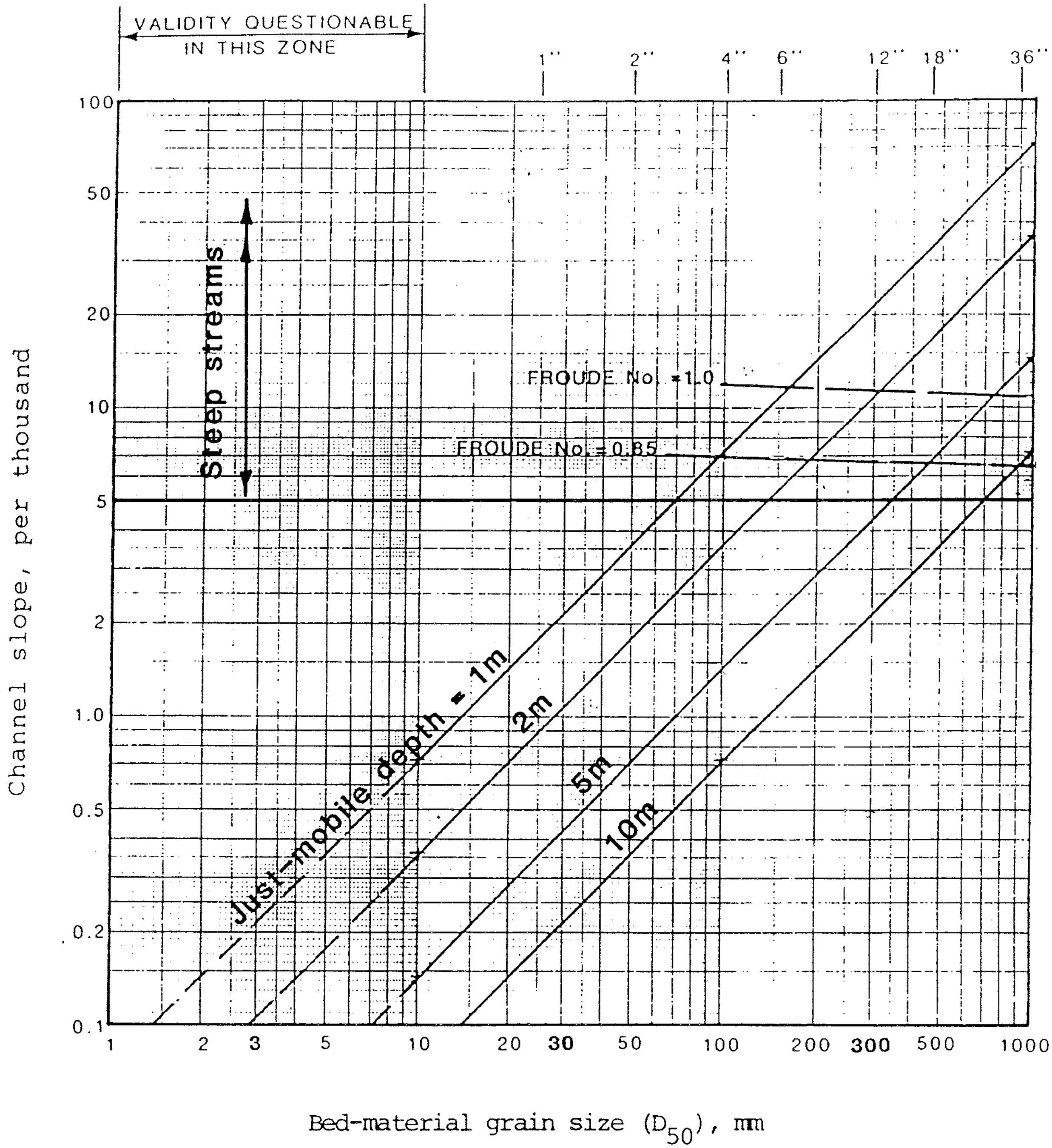


Figure 1. Relationships of slope, grainsize, depth and Froude No. for just-mobile channels

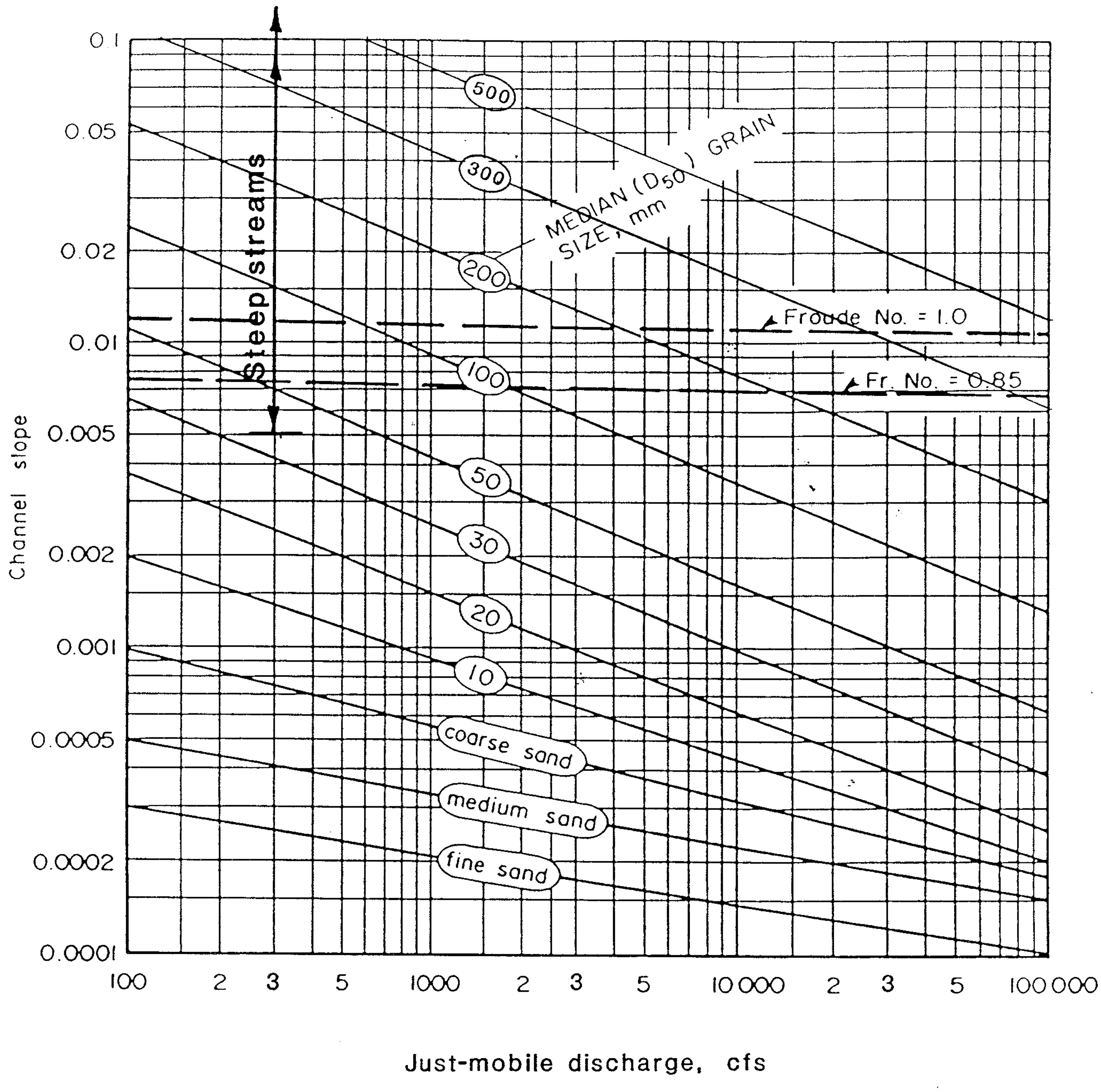


Figure 2. Relationships of slope, grainsize, discharge and Froude No. for just-mobile channels

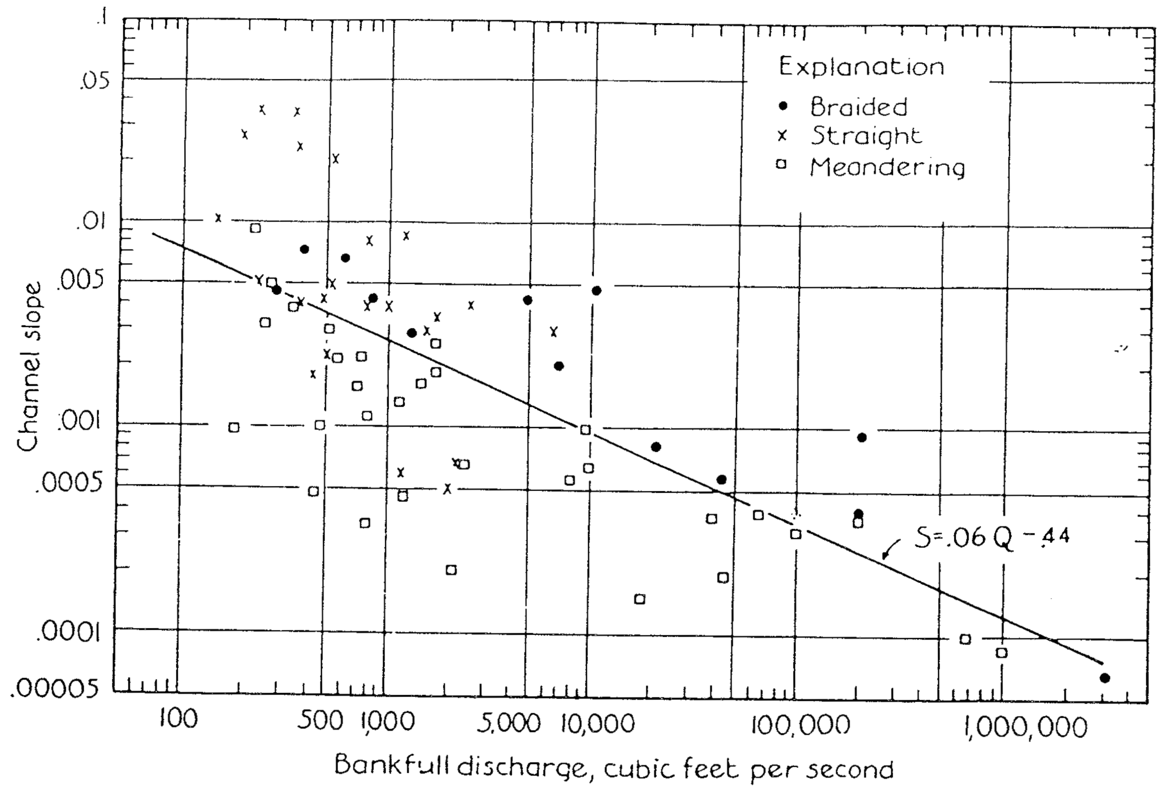


Figure 3. Slope–discharge diagram by Leopold et al (1964) with lower limit for braided channels

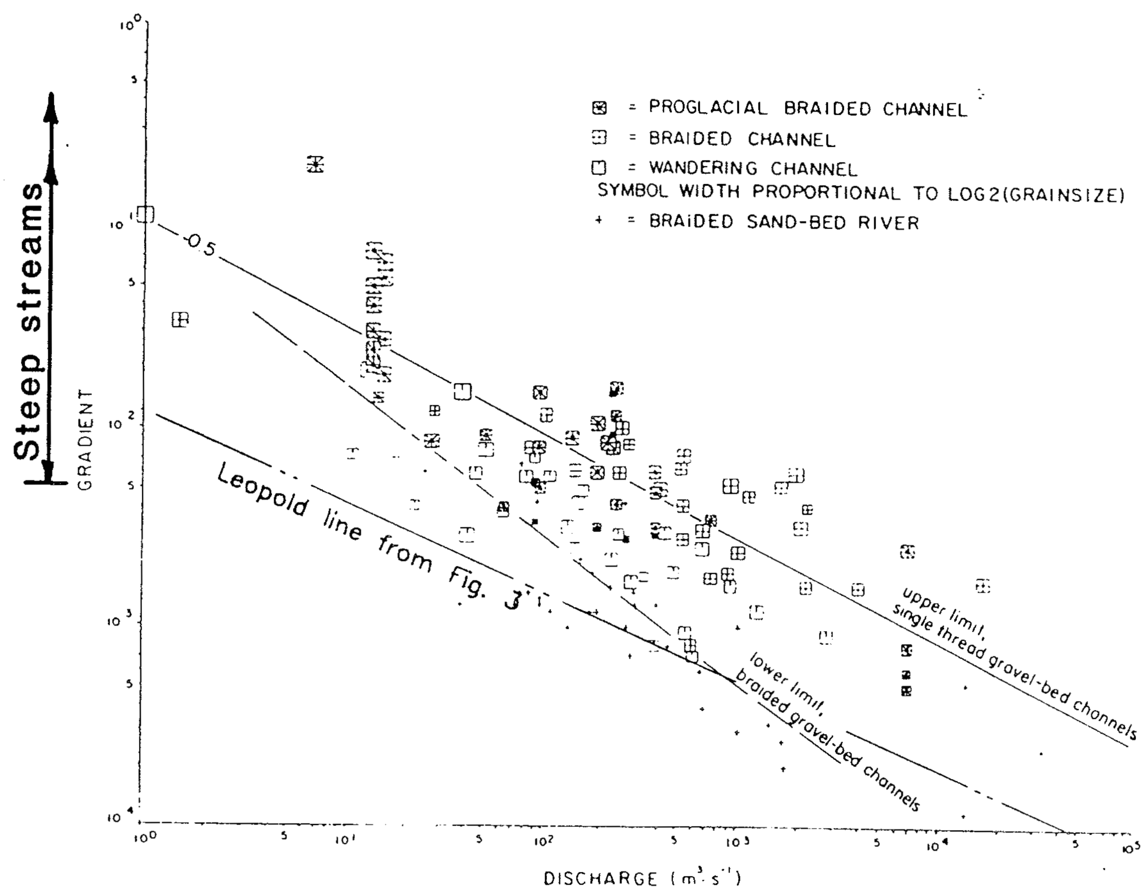


Figure 4. Similar diagram by Church and Kellerhals (1989)

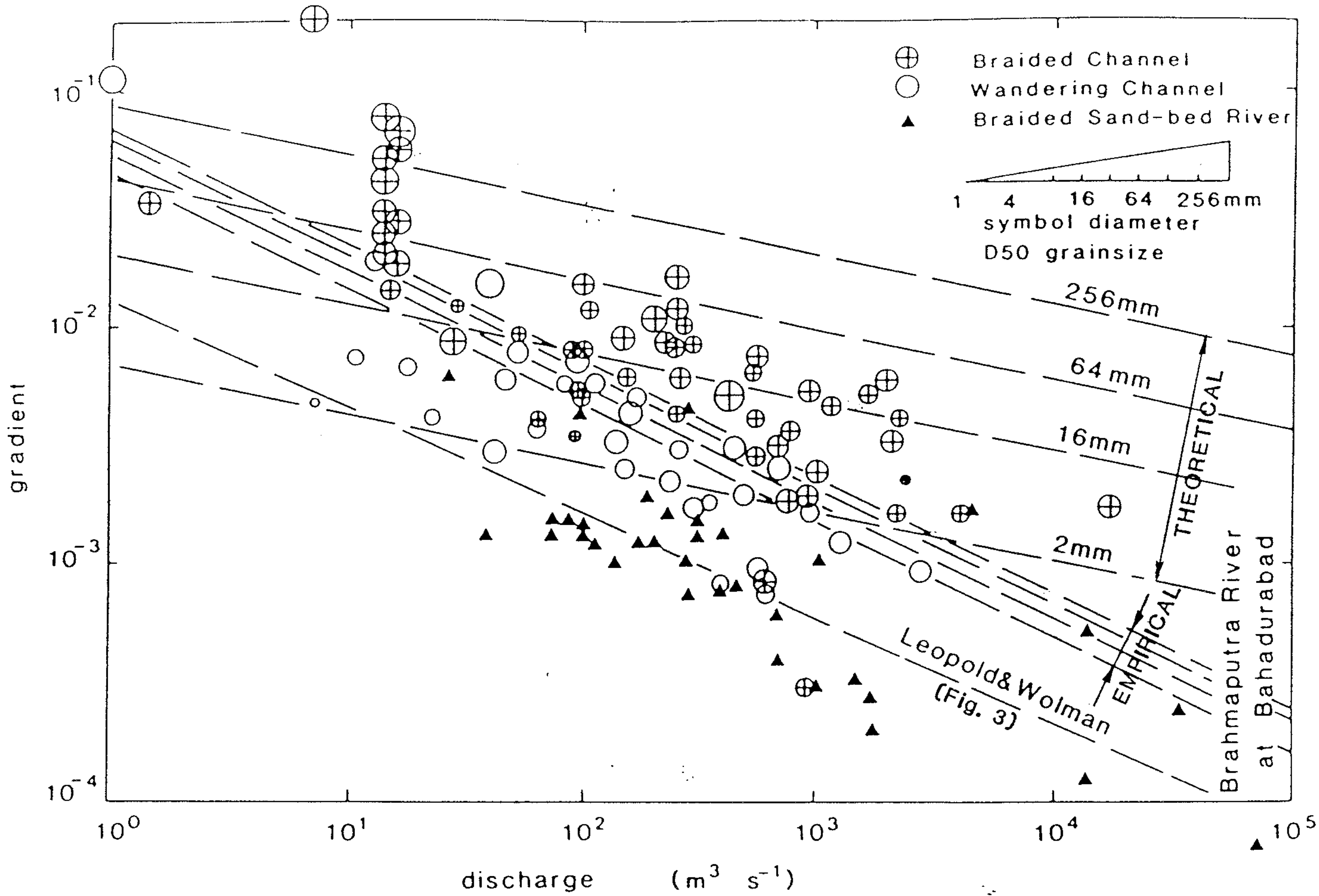


Figure 5. Slope–discharge–grainsize plot by Ferguson (1984) indicating lower limits for braided channels

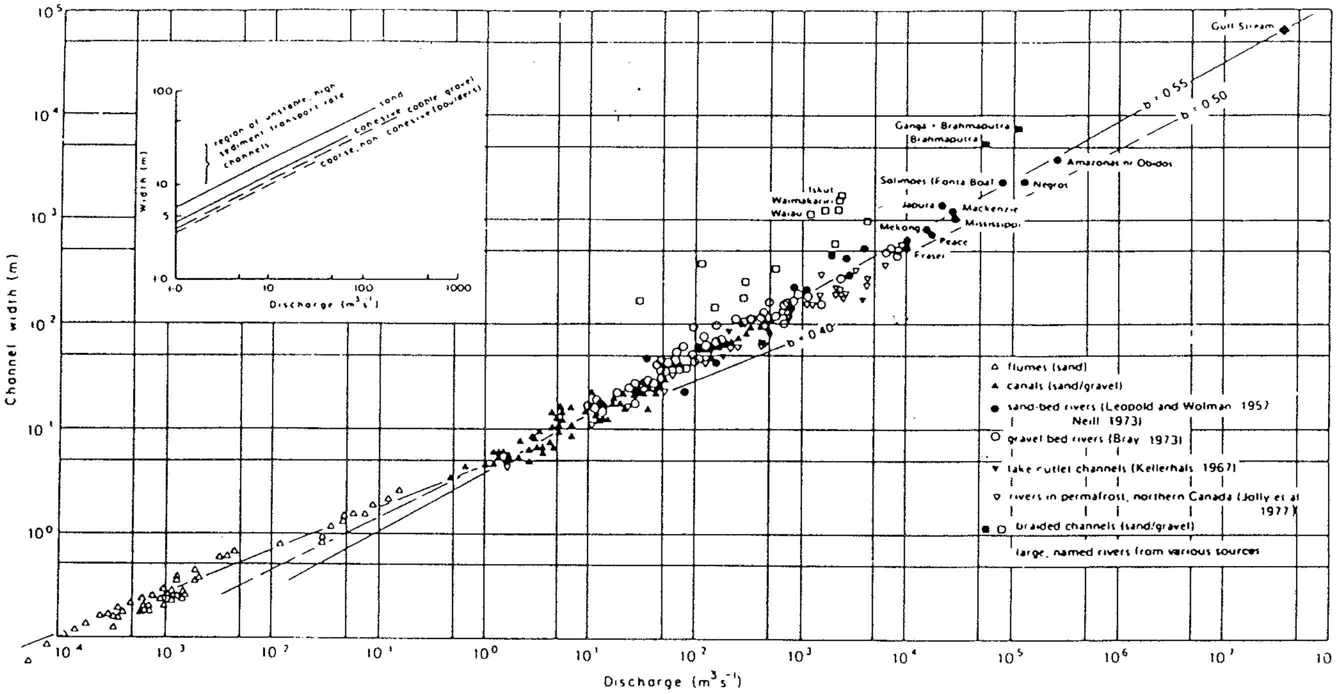


Figure 6. Comprehensive width–discharge plot by Kellerhals and Church (1989)

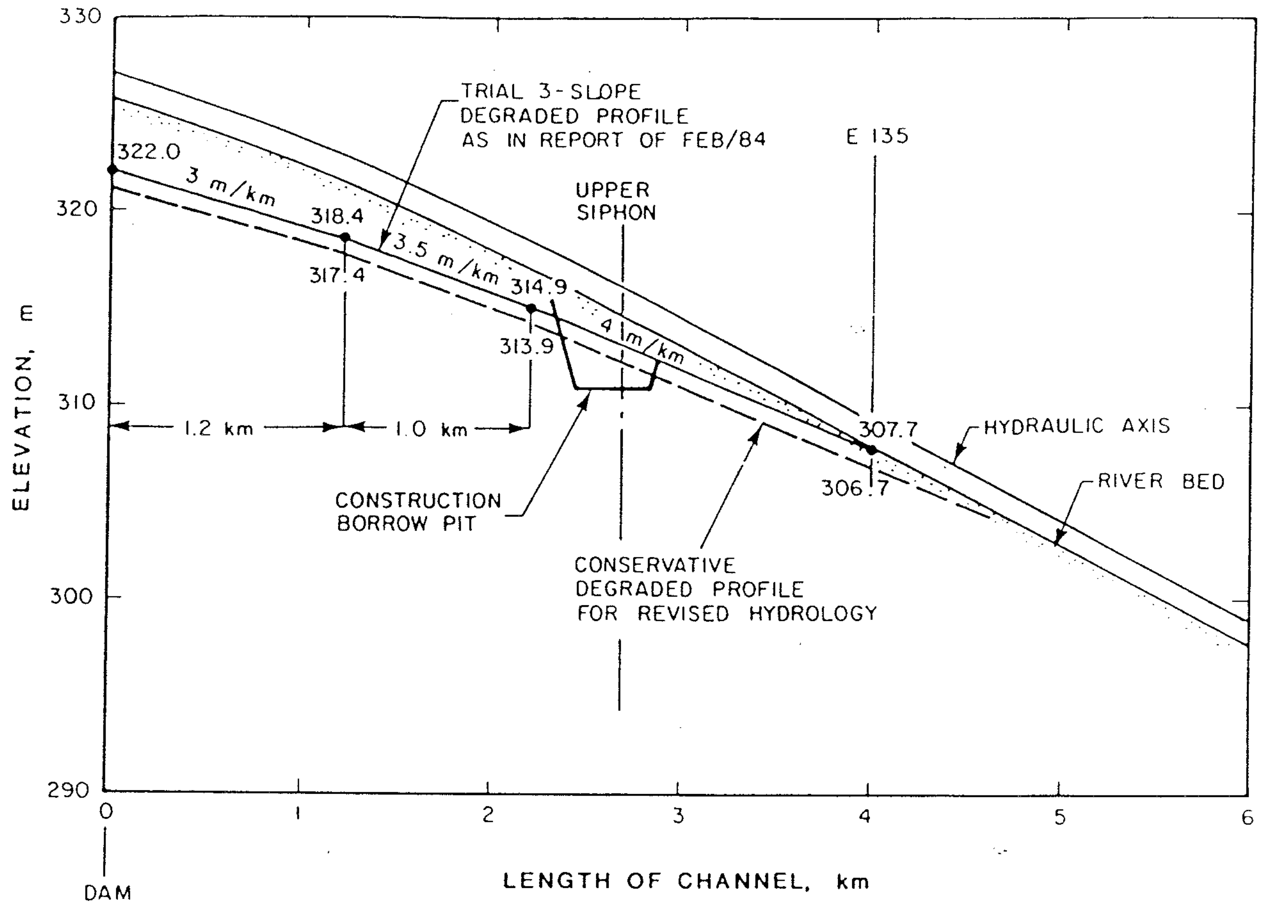


Figure 7. Predicted 50-year degradation below dam on Rio Maule

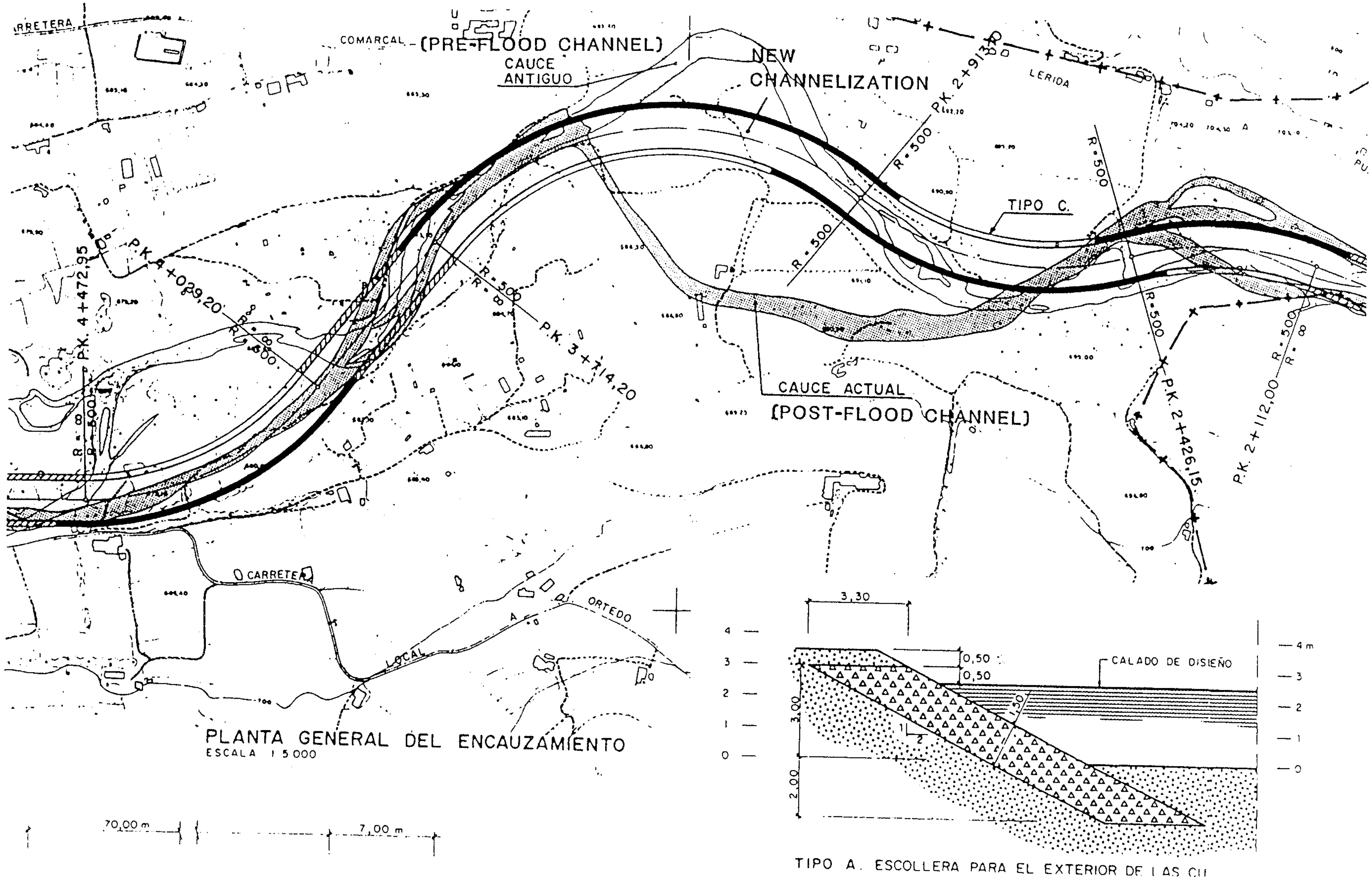


Figure 8. Partial plan and section of Rio Segre Channelization

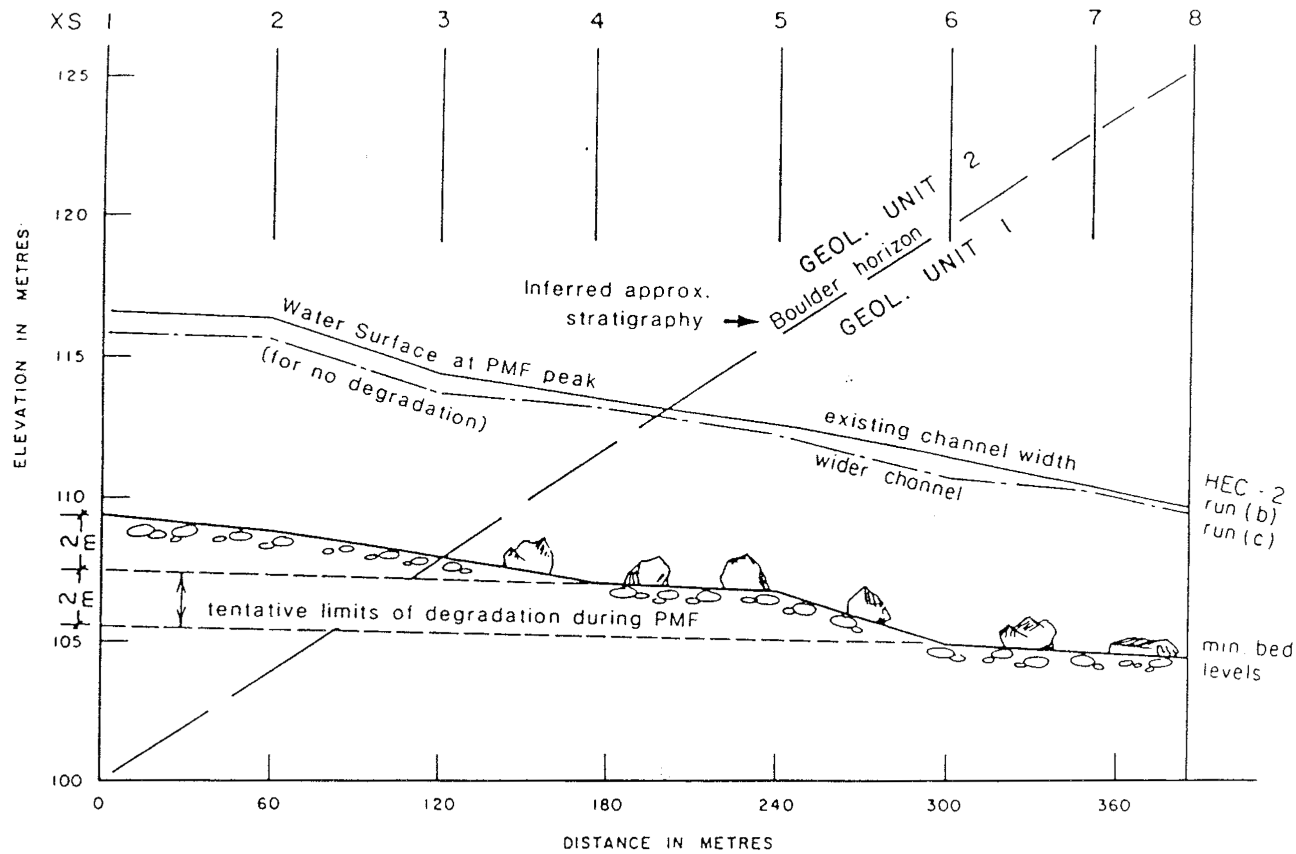
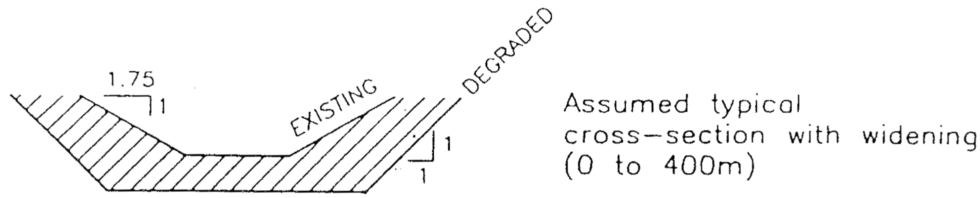


Figure 9. Predicted PMF degradation of Channel "X"



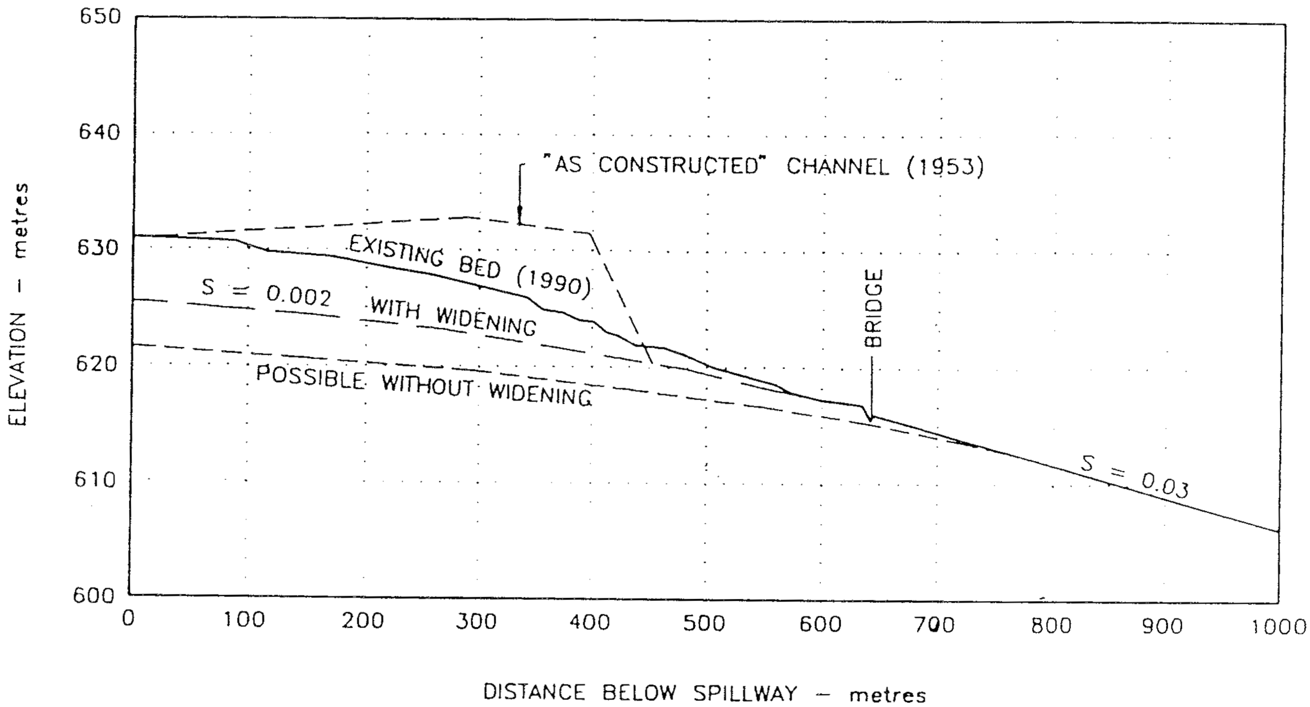


Figure 10. Predicted PMF degradation of Channel "Y"

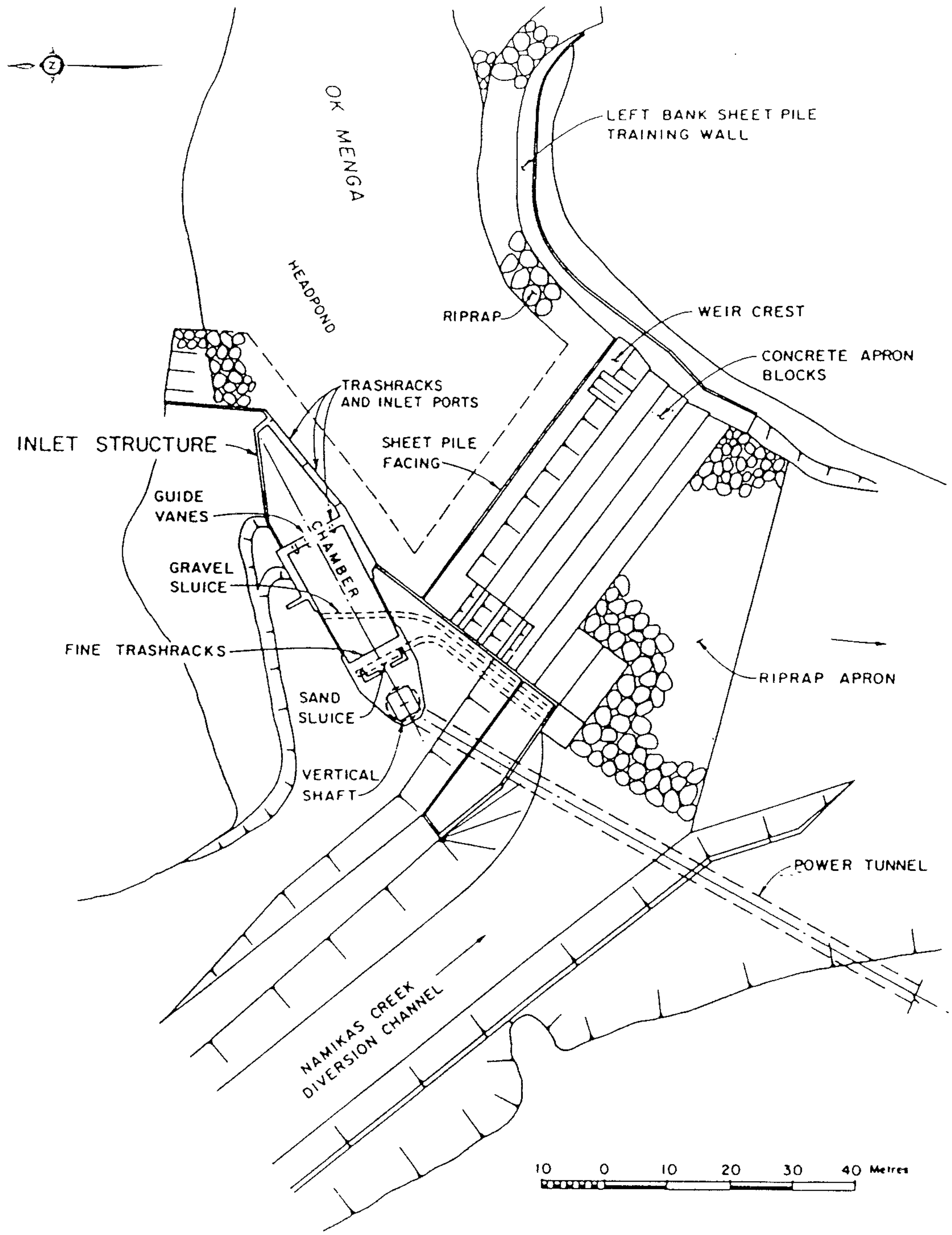


Figure 11. Plan of intake works on Ok Menga
(McCreath et al, 1990)

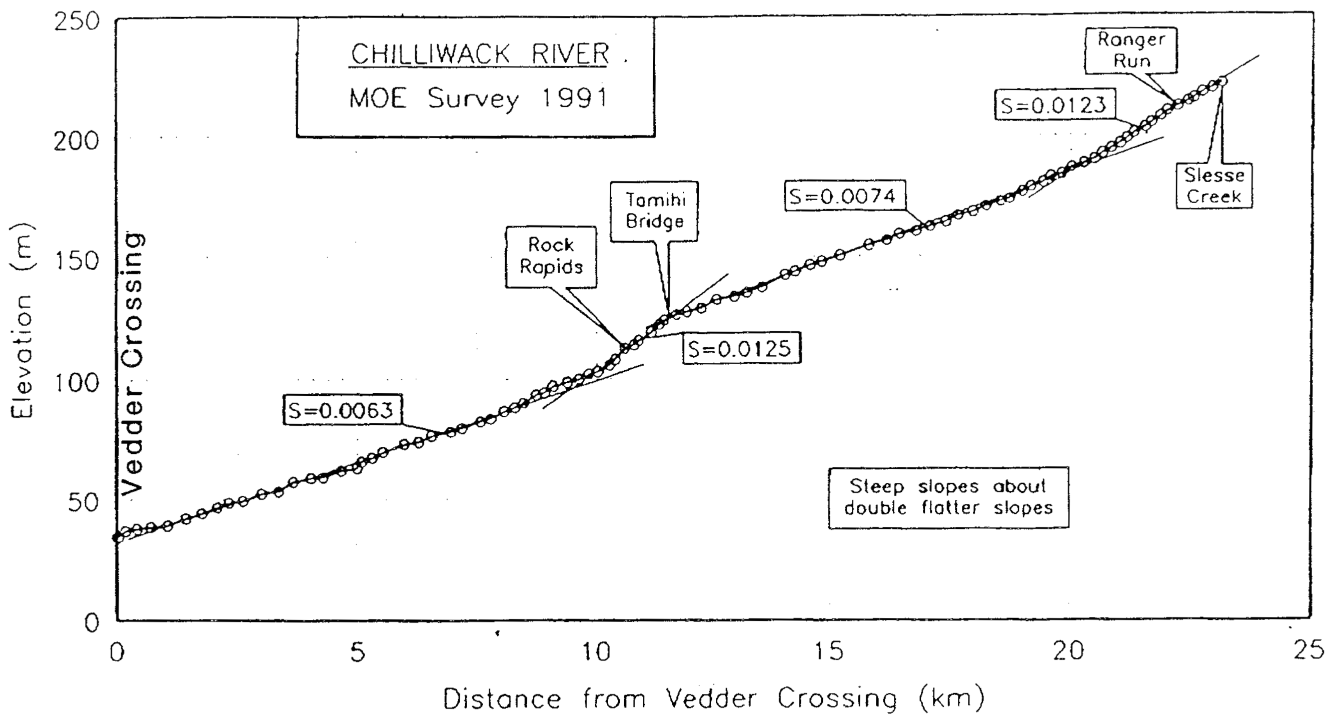


Figure 12. Longitudinal profile of Chilliwack River

Why Steep Channels?

by

William A. Thomas[1]

1. **Background.** In 1989 the Flood Control Channels Research Program was established to either assemble or develop guidance for the hydraulic design of channels and disseminate it to district and division offices. A nation-wide survey of flood control channels was conducted which served as the basis for planning the R&D work. Based on that survey, a critical review by the Committee on Channel Stabilization, and various contacts we made with districts, divisions and OCE, the following work units were proposed

Table 1. Research Work Units Proposed in 1989

*1) Controlling Stream Response to Channel Modification
*2) River Bend Erosion
*3) Hydraulic Losses in Natural Channels
*4) Sediment Transport in Small Channels
5) Composite Channel Analysis
6) Channel Investigations
7) Channel Response and Channel Forming Q
8) SAM Design Simulator
*9) Gravel and Boulder Rivers
10) Spatially varied Flow
11) Supercritical Flow Channels
12) System Sediment Loading and Delivery
13) Hydraulic Routing Techniques
14) Behavior of Sand Bed Streams on Steep Slopes

Note: * work units selected for funding in FY 1989

We presented the program to our Field Review Group who endorsed the first 4 work units. However, they voted to move the

[1] Research Hydraulic Engineer, US Army Engineer Waterways Experiment Station, Hydraulics Laboratory, Waterways Division. This paper is for presentation at the Steep Streams Workshop, Seattle, WA, 27-29 October 1992.

Gravel and Boulder Rivers work unit to priority 5 so it could begin the first year rather than wait for out-year funding. Maj Monte Pearson was put in charge of that work.

2. **Initial Tasks.** The initial task was to assemble information on gravel and cobble bed rivers that would allow us to establish the limits of applicability of existing technology. That technology includes regime analysis, sediment sampling equipment, sediment sampling procedures, sediment transport theory, calculation of water surface profiles, and numerical modeling of water and sediment movement. One effort focused on understanding the physics of the river mechanics and sedimentation processes which develop bedform morphology. Another effort was to apply field reconnaissance forms being developed for sand bed channels to gravel and boulder rivers. A third effort was to identify and make contact with others doing research on this topic. A fourth effort was to locate data. A companion effort to these was to establish contacts with district and division offices who were designing channel projects on gravel and cobble bed streams.

3. **Steep Streams Workshop.** This workshop is a milestone in pursuing these several tasks. Particularly, it brings together some of the districts who are designing projects, other researchers on this topic, and our findings to date.

What is a "steep" stream. Intuitively, one thinks of a mountain stream having an abundance of geologic controls, rapids, pools, and boulders. Trees line the banks. The gradient decreases slightly and the stream bed becomes covered with gravel and cobbles. Runoff consists of a mixture of rainfall and snow melt events. The more extreme events are flashy and short lived. The gradient is stable because the large particle sizes resist movement until the spring freshet, or some other large event, occurs.

There is another type of streams, however, that has a steep gradient because its inflowing sediment concentration is so high the slope must be steep to transport the inflowing load. A sand bed arroyos on an alluvial fan is an example. Although this would probably qualify as "steep" using a slope criteria, it does not conform to the characteristics of the gravel/cobble bed stream listed above. This workshop will emphasize the gravel/cobble bed streams.

4. **Present Technology for Sand Bed Streams.** The emphasis in the Flood Control Channels Research Program during the past 4 years has been to provide systematic methods that hydraulic design engineers can use to calculate four of the six Hydraulic Design Parameters for a channel. The six parameters are shown in table 2.

4.1 Hydraulic Design Parameters. The hub of the computations is a PC-program called SAM. That stands for "Hydraulic Design Package for Channels." The purpose of SAM is to provide hydraulic design engineers with a method, based on state of the art theory, for rapidly calculating stable channel dimensions in both fixed and mobile boundary streams.

Table 2. Six Hydraulic Design Parameters

*1.	WIDTH
*2.	DEPTH
*3.	SLOPE
*4.	HYDRAULIC ROUGHNESS
5.	PLAN-FORM
6.	BANK-LINE MOVEMENT

Note: * Marks parameters calculated by SAM

4.2 <u>Organization of SAM.</u> The three major calculation modules are

HYDRAULIC CALCULATIONS	**SEDIMENT TRANSPORT CALCULATIONS**	**SEDIMENT YIELD CALCULATIONS**

These are supported by

PSAM - Input Data Screens programmed in QUICK-BASIC

HECDSS - Data storage and graphics

SAM.t95 - a utility that reads HEC2 TAPE95 and calculates average hydraulic properties for sediment transport calculations.

SAM.aid - guidance for selecting a transport function

PARTICLE SETTLING VELOCITY - CORPS program H0910 for calculating the settling velocity of sediment particles.

ENDOW - an Expert System that includes Environmental Engineering considerations in channel design.

4.3 <u>HYDRAULIC CALCULATIONS.</u> SAM.hyd is a normal depth solution of the Manning Equation using either simple of complex cross sections. It calculates hydraulic roughness in each panel in the cross section (a panel is the space between 2 consecutive coordinate points on a cross section plot.):

Manning n-value

Colebrook-White Diagram (Keulegan Velocity Distribution equations)

Strickler Equation

Limerinos Equation

SCS Grass Equations for 5 Types of Grass

4.4 <u>Options for Solving Manning's Equation.</u> Any of the variables can become the dependent variable except for side slope, z.

$$Q = f(D, n, W, z, S) \tag{2.1}$$

where

Q = water discharge
D = water depth
n = n-value
W = bottom width
z = side slopes of the channel
S = energy slope

<u>Water discharge</u> will be calculated when all variables except water discharge are prescribed.

Normal Depth will be calculated when all variables except Water Surface elevation are prescribed.

Bottom Width of the channel will be calculated when all variables except bottom width are prescribed.

Hydraulic Roughness (n-values and Ks values) will be calculated when all variables except n-value are prescribed.

Energy Slope will be calculated when all variables except slope are prescribed.

Flow Distribution will be calculated when all variables are prescribed and during any other computation.

Bed Roughness Prediction is made with Brownlie's method.

Composite Hydraulic radius and n-value can be calculated by four options:

(1) **Alpha method,**
(2) **Equal Velocity method,**
(3) **Total Force method and**
(4) **Conveyance method.**

Riprap Size is calculated if the bed shear stress is greater than Shield's critical value.

Effective Width, Depth and Velocity are calculated for use in sediment transport calculations.

4.5 Options for Calculating Stable Channel Dimensions. Stable channel dimensions refer to combinations of width, depth and slope for which the resulting hydraulic parameters will transport the inflowing bed material load.

Channel Width, Depth and Slope are printed for a wide range of values.

Regime Width is calculated for the prescribed water discharge.

Minimum Stream Power criteria is imposed and the resulting width, depth and slope are printed.

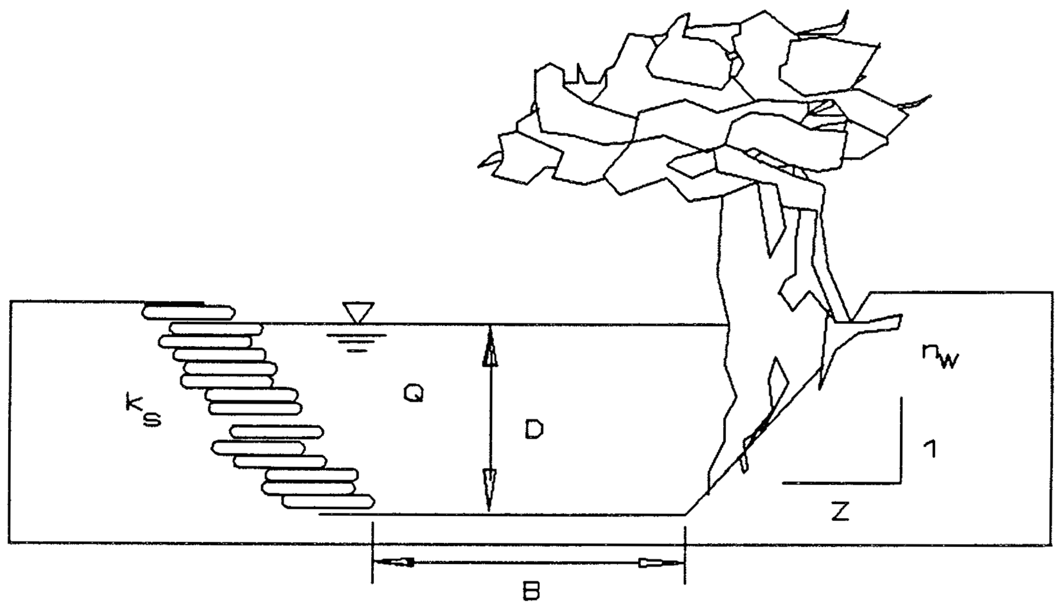


Figure 1. Typical Cross-Section used in Analytical Method.

4.6 <u>Plots.</u> The following hydraulic plots are valuable for decisions about sand bed channels.

Table 3. Plots of Hydraulic Parameters

STAGE VERSUS N-VALUE
STAGE VERSUS AVERAGE VELOCITY
STAGE VERSUS EFFECTIVE VELOCITY
DISCHARGE VS EFFECTIVE VELOCITY
STAGE VERSUS DISCHARGE
STAGE VERSUS WIDTH
STAGE VERSUS EFFECTIVE WIDTH
STAGE VERSUS HYDRAULIC RADIUS
CROSS SECTION GEOMETRY
SLOPE VERSUS WIDTH

4.7 <u>Riprap Size.</u> Riprap size is calculated using the Maynard equations. The 12-standard riprap sizes shown in EM 1110-2-1601 are coded into SAM In addition, up to 5 sizes of quarry run riprap can be encoded. Shield's Diagram is used to determine if the stream bed particles are stable or if riprap is needed.

5. **Sediment Transport Calculations.** The purpose of this module is to provide a sediment transport function for the range of field conditions where projects will be developed.

5.1. <u>Available Sediment Transport Functions.</u> The following have been incorporated into SAM.sed. Except for Brownlie, these are functions from HEC-6.

Table 4. SEDIMENT TRANSPORT FUNCTIONS

ACKER-WHITE.
ACKER-WHITE, D50
BROWNLIE
COLBY
LAURSEN(COPELAND)
LAURSEN(MADDEN),1985
MPM(1948).
MPM(1948),D50
TOFFALETI.
TOFFALETI-MPM
TOFFALETI-SCHOKLITSC
YANG.
YANG,D50

5.2 SAM.aid - Guidance in transport function selection. A computer program has been developed search existing data sets and identify the most appropriate sediment transport function given values for the following:

(VELOCITY, DEPTH, SLOPE, WIDTH, AND D_{50})

First, test data sets that match the screening parameters of the project are identified. Then, each data set is inspected to determine which transport functions did the best in reconstituting the measured transport rates. The functions are then ranked from best to worst and the best three are listed.

6. **Sediment Yield.** The purpose of "Sediment Yield" is to calculate the weight and volume of sediment passing a cross-section during a specified period of time. Typically that is annually, but it can be a single event.

FLOW DURATION CURVE method and

FLOW HYDROGRAPH method.

7. **Basic Philosophy.** The effort has been directed toward packaging existing technology so design engineers can be sure their projects will be save, reliable, maintainable, and environmentally pleasing. Procedures meet the requirements of Engineering Regulations and meet the requirements in the water resources development legislation. The methods have been packaged for MS-DOS computers.

8. **Points of Discussion.** The technology in SAM is for sand bed streams. It solves the Manning equation in the usual way; calculates bed shear stress; uses the Shield criteria; calculates sediment transport; and calculates sediment yield. We are using existing equations. How far can those equations be pushed?

Regime equations have been used for some time by Charlie Neill and his colleagues in Canada. Some of their streams have very coarse bed sediment. How far can those equations be pushed?

How does one sample a coarse bed stream?

How does one obtain a suitable value for the wash load and the "through-put" load without extensive data collection?

What are the major gaps in knowledge that need to be filled before applying SAM to steep gradient streams?

A Review of Natural-Gravel-Transport-Detection Experiments at Squaw Creek, Montana, 1981-1991

Stephan G. Custer, Department of Earth Sciences, Montana State University, Bozeman, MT 59717-0348

Abstract

The magnetism of natural particles is a useful tracer for the study of gravel transport on stream bottoms. During the past 10 years a device has been developed at the Technische Fachhochschule, and the Freien Universität, Berlin, Germany and Montana State University which detects the motion of naturally magnetic pebbles and cobbles on the stream bottom. Since the tracer does not have to be installed, assessment of transport is possible under undisturbed natural conditions. The detector system counts particles as they move over the sensor. The records provide insight into the spatial and temporal distribution of bed-load transport in natural streams. Data from this detector, bed-load sampling, and hydraulic measurements suggest that the threshold for transport varies in response to geomorphic as well as hydraulic processes. Transport peaks do not necessarily correspond to discharge peaks. The transport rate varies both temporally and spatially across the stream bottom. The variability suggests selective transport of gravel particles in response to geomorphic as well as hydraulic variables.

The detector will work in many gravel-bed streams including those in the Pacific Northwest,and Alaska. The detector has potential as a sensing device to alert managers of the onset of particle motion and its location. The device also has potential to further elucidate gravel-transport processes and the factors which control the threshold of gravel motion, transport location, and the temporal distribution of transport particularly if the universal problems associated with physical sampling during gravel transport can be solved or at least reduced during calibration.

A Review of Natural-Gravel-Transport-Detection Experiments at Squaw Creek, Montana, 1981-1991

Introduction

The Problem

Bed-load transport is of interest to people planning, constructing, and operating engineering structures in gravel-bed rivers (Neil and Hey, 1982; Wang, and others, 1987; Ackers and Thompson, 1987; Griffiths, 1989; Lagasse, and others, 1991; Richardson and others, 1991). Questions include: What is the threshold for motion of bed material? How much material can be expected to move? How much material is moving? Where on the bed is material moving? These questions are difficult to answer because bed-load transport is not typically visible, and because of difficulties with hydraulic measurement and bed-load sampling under conditions favorable for coarse-gravel motion. A variety of tracers have been used to explore gravel transport (Bunte and Ergenzinger, 1989). In 1981, Ergenzinger and Custer (1983) developed a measurement technique which uses the natural magnetism of particles in a stream bed to detect transport. During the past 10 years, the technique has evolved, and has provided insight into transport processes in gravel-bed systems. This paper reviews the development of the technique and some results from measurements at Squaw Creek, Montana. The purpose of the review is to identify the advantages and disadvantages of the method, illustrate some applications, and provide some insight into the nature of gravel-transport processes in natural streams.

The Site

The research site is 50 m upstream of the confluence of Squaw Creek and the Gallatin River in Gallatin County Montana (Figure 1). The site characteristics have been summarized by Ergenzinger and Custer (1983) and by Bunte and others (1987). The stream drains an area of 106 km^2, has a relief of 1520 m, and a Strahler stream order of 4. The bedrock which underlies the drainage basin is approximately 25 percent Archean quartzofeldspathic gneiss, 20 percent Paleozoic and Mesozoic sediments, and 55 percent Eocene andesitic and basaltic lava flows, mudflows, and intrusions (McMannis and Chadwick, 1964). The 50 year recurrence interval discharge is 20 m^3/s (Ergenzinger and Custer, 1983). The bankfull discharge ($Q_{1.5}$) is 5 m^3/s. Water-surface slope and bed slope vary, but are approximately 0.02 (Bugosh, 1988). The stream peaks between 1 May and 1 July. The peaks are typically caused by snow melt, but some rain-generated peaks in the absence of snow have been observed.

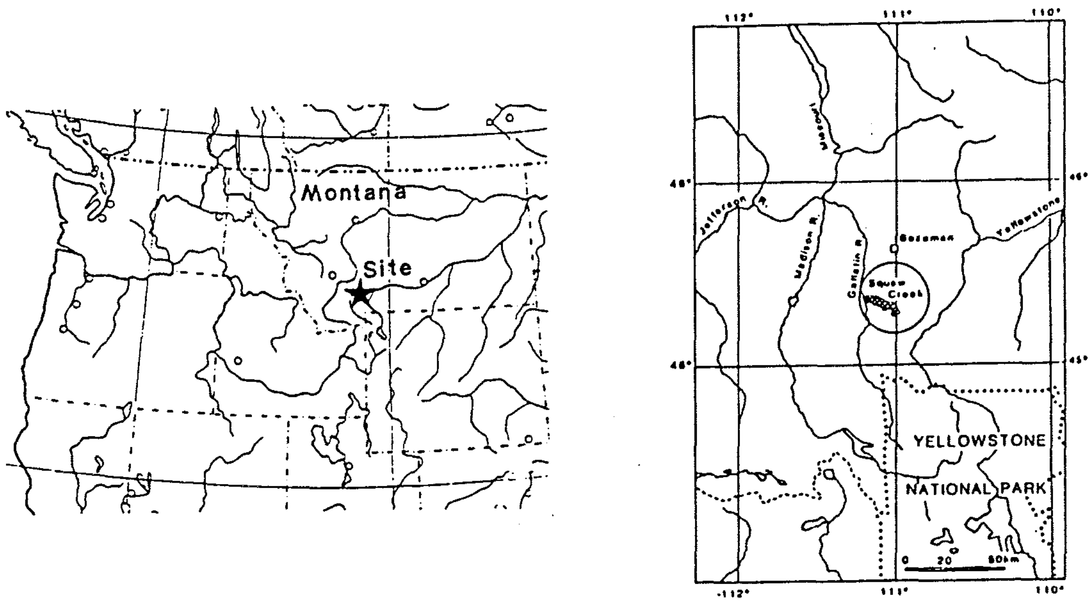


Figure 1. Location map for Squaw Creek (after Bugosh and Custer, 1989 and Bunte and others, 1987).

The site is divided into two channels (Figure 2). The main (south) channel is approximately 11 m wide. The north channel is approximately 6 m wide. The channels are separated by a gravel bar. The east end of the gravel bar is stabilized by an alder. A road bridge to the Castle Rock Baptist Bible Camp across Squaw Creek constricts the channel to a width of approximately 8 m. When the U.S. Forest service installed the bridge, logs with wire mesh draped below them were placed in the stream bed, and attached to the bridge abutment. These logs were necessary because Squaw Creek is graded to the Gallatin River and displays considerable dynamic behavior in response to rising and falling stage of the Gallatin River (temporary local base level). Each year Squaw Creek deposits gravel at the confluence with the Gallatin River during high flow, and cuts through the deposit in early summer when stage on the Gallatin River falls.

The channel at the site is dynamic. When research began in 1981, there was a single south channel with the thalweg directed approximately normal to the bridge and an abandoned channel with young vegetation north of discharge-bearing channel (Figure 2). In the spring of 1981, a 16 m^3/s flood caused erosion and deposition which split the channel into two parts. In 1983, a log jam burst and the north channel was rearranged again and returned to a single thread. The single thread configuration was maintained until 1990 when flood waters began to encroach on the abandoned north channel. Although the stream changes its channel configuration periodically, the channel is responding to naturally varying local conditions above the bridge. The creek has remained within the natural terrace banks on either side of the stream throughout the study period. In this sense, the channel position appears to be stable.

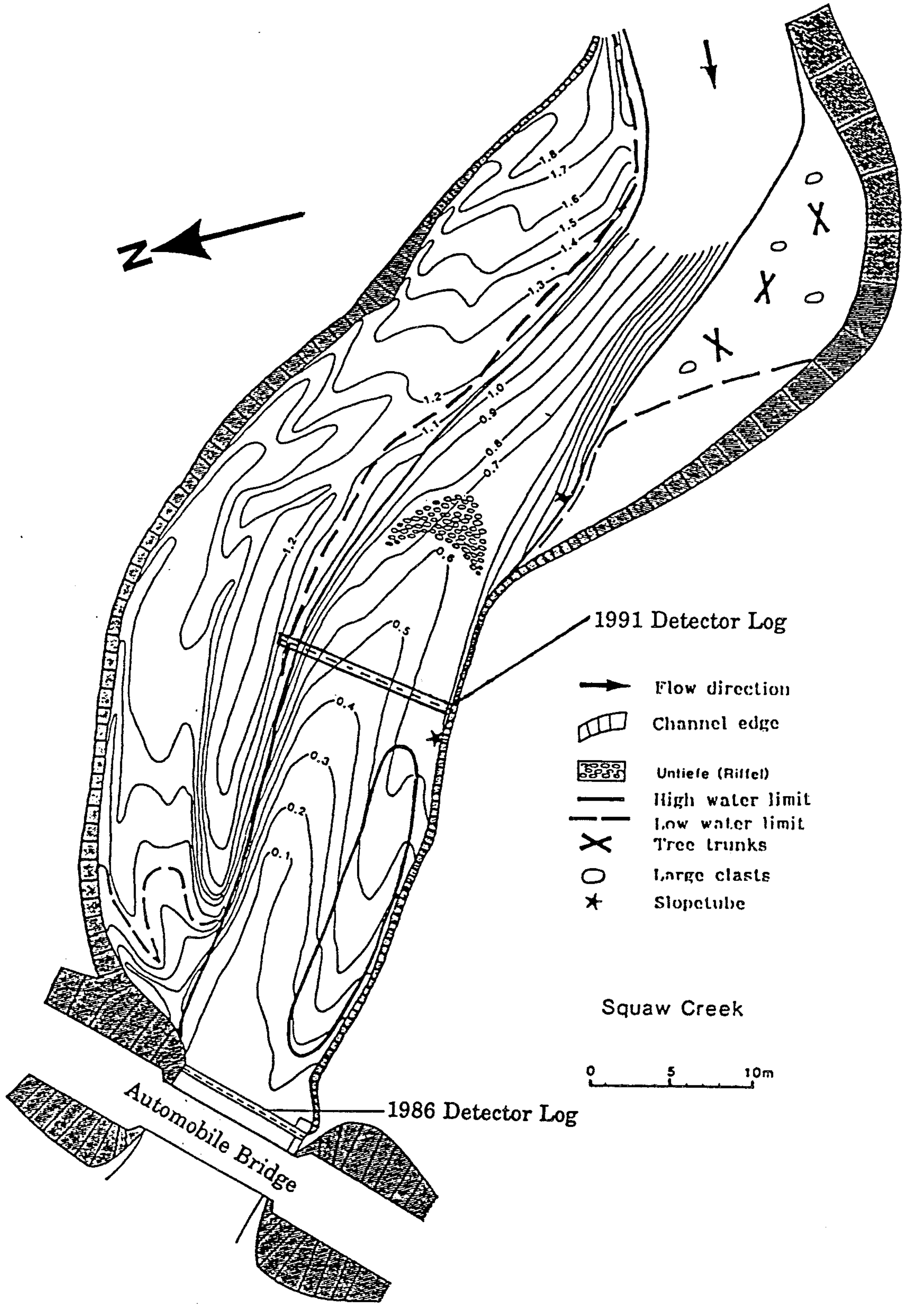


Figure 2. Site Map (Modified from Bunte, 1991, Figure 3.07)

Several investigators have measured the particle size distribution of material in the gravel bars and channel of the stream (Bunte and others, 1987; Custer and others, 1987; Bugosh 1988; Bunte, 1991) (Table 1). Median particle size on the bar is approximately 19-24 mm and in the channel adjacent to the bar in the thalweg is 100-140 mm. More than 16 percent of the particles in the channel are larger than 200 mm. Material as large as 180 mm has been trapped in bed-load nets at the bridge.

Table 1. Example cumulative particle size distribution (mm) (Bunte, 1991)

	d_5	d_{16}	d_{25}	d_{50}	d_{75}	d_{84}	d_{95}
bar 1	0.5	2.3	7	19	32	40	67
bar 2	0.4	1.8	5	24	60	90	200
channel	0.5	2.8	18	140	190	200	220

Work began on bed-load measurement at the Squaw Creek site in 1980-81 (Ergenzinger and Custer, 1983). At that time there was a single automobile bridge across the stream at the site. Over the years, walk ways at the Bible-Camp bridge have been expanded, and new measurement platforms upstream of the original site have been installed. A 15 cm "catwalk" was installed in 1981 under the road bridge. This "catwalk" and was moved up stream to the detector log in 1983. A 1 m "suspension bridge was installed just upstream of the "catwalk" in 1986, and in 1990 a second 1 m-wide measuring platform was installed approximately 20 m upstream of the road bridge.

The Instrument

<u>The Detector</u>

The gravel-bed-load-motion detector operates on electromagnetic principles. The idea for natural-particle detection arose from artificial electromagnetic tracer techniques which used a magnet epoxied into holes drilled in cobbles (Ergenzinger and Conrady, 1982). The magnetized cobbles were placed on or in the stream bed. In the artificial-magnet approach, motion of the artificially magnetized cobble induces a voltage in a coil above the stream. The voltage is detected. The signal is produced by induction. No input power is needed for the detector, but power is needed for the amplifiers and recorders. At Squaw Creek, instead of using an artificial magnet, naturally magnetic pebbles and cobbles are used as the tracer, and because the magnetism of rock is often weaker than that of an artificial magnet, the coils which detect the motion are buried in the stream bottom closer to where the motion takes place (Ergenzinger and Custer, 1983). The advantage of the use of naturally-magnetic gravel as a tracer is that the material need not be moved from its natural position in the bed (a necessary problem with artificial tracers), and the number of tracers in the stream bed is not constrained by the time and patience of an assistant with the tedious responsibility to insert tracers in the gravel particles. Thus, the technique detects motion of many naturally-magnetic-gravel particles from natural position on the stream bottom. A brief history of the evolution of the detector and recorder provides insight into the principle of operation, the current state of the art of the detector, and the limitations of the detector.

The detectors have become smaller and the recorders have introduced better amplification, filtering, and data acquisition (Figure 3). The 1981 detector consisted of four 20-cm-long coils spaced 14 cm apart and connected in series (Ergenzinger and Custer, 1983) (Figure 3 A and B). . Each coil consisted of 9000 windings of 0.2 mm copper wire. The windings were around a 1-m-long , 2 cm diameter iron bar. The detector was placed in a channel in cast concrete and surrounded by roofing tar. The concrete block was 1.25 m long, 0.2 m wide and 0.15 m high. A voltage was produced when a naturally magnetic particle moved across the detector. The detector worked well, but was heavy (approximately 90 kg), required elaborate wiring protection against cobble impact, and was eroded from its position, washed down stream and buried in a gravel bar during an 18-year recurrence-interval flood (Ergenzinger and Custer 1983; Bunte, 1991). To make the detector more manageable and sensitive, smaller coils with approximately 20,000 windings were built (Custer and others, 1987) (not shown in figure 3). Instead of burying the detector against the log that maintains base level at the bridge, the 30 cm coils were encased either in 7.6 cm diameter concrete cylinders or in 4.5-cm-high resin half cylinders. The cylinders and half cylinders were bolted to a log installed across the stream and attached to the base-level-control log. In addition to the problems with line noise from the power supply to the amplifier and lightning strikes, additional noise was introduced from the physical motion of the detectors through the Earth's magnetic field (Custer and others, 1987). The smaller coils were too light and could not be attached to a log sufficiently tightly to eliminate vibration. The problem of wire protection remained.

The early coils were oriented with the core and axis of the windings parallel to the stream bottom. In 1986, a new design was implemented with the core and axis of the windings perpendicular to the stream bottom to produce a stronger signal from smaller sensors (Spieker and Ergenzinger, 1990; Ergenzinger and others, 1992) (Figure 3 C and D).

> "Each detector unit has a length of 1.4 m and consists of over 300 chokes ..." (7 x 7 mm wire coils wrapped around a ferrite core.) " ... The axes of the chokes are perpendicular to the river bottom. The chokes of each detector are serially connected so that the total inductivity of each detector is about 21 Henry (V s A^{-1})" (Spieker and Ergenzinger, 1990, p. 170).

The detectors are surrounded by a plastic channel and fixed with resin into "dove-tail" grooves in a 15 x 25 cm x 8 m log imbedded in the stream against the log which

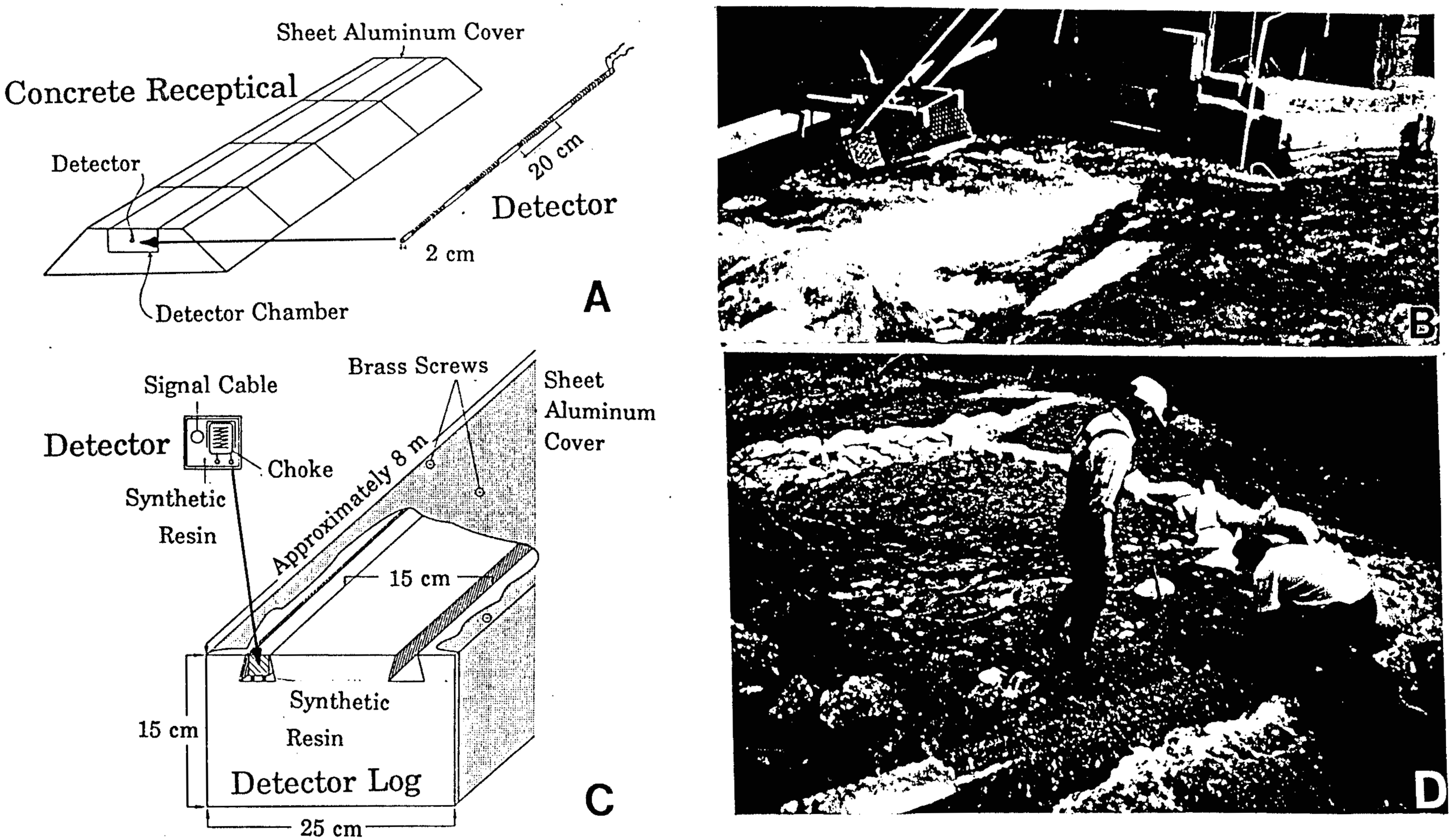


Figure 3. Detector evolution. A) schematic of 1981 detector and holder (after Ergenzinger and Custer, 1983; B) detector in stream, 1981 (note white tubes containing the wires at bridge abutment (after Custer and others, 1987); C) schematic of 1986 detector (from Spieker and Ergenzinger, 1990); D) 1986-type detector in place at the upstream location (Figure 2) with part of aluminum cover absent due to cobble impact.

maintains base level under the road bridge. The great weight of the water-logged detector provides stability, the multiple blocks provide opportunities to better identifytransport location. The two detector strips were designed to estimate time of passageof waves and particles. The log was covered with 0.8 mm (0.0315 in.) thick aluminum sheet to protect the detectors from physical damage due to cobble impact and to provide a smooth surface which does not accumulate bed material. Thinner aluminum flashing (0.292 mm; 0.0115 in.) was used on a second detector installed 28 m up-stream of the first in 1990 (Figure 2), and this cover was destroyed by cobble impact in one season (Figure 3 D) while the thicker aluminum remained in tact. The thicker aluminum covering is desirable. Because the wires for the detector are inside the detector capsule in the dove-tail groove, they are protected from physical damage (Figure 3 C). In the old design wire conduit and wires were damaged annually (Figure 3 B). Since 1986, the same detector design has been used.

The physical characteristics of the detector are important because of noise reduction considerations (Custer and others, 1987) particularly in the later-generation detectors which amplify the signal up to 10^{+6} times. The following noise-reduction considerations are essential: 1) As few ferric nails and bolts as possible should be used. Brass and aluminum fasteners and shields are desirable because they do not corrode and do not generate a signal if they vibrate. 2) The detector itself must not move or vibrate even slightly because motion of the detector coils through the magnetic field of Earth produces a signal. The detector should be attached very securely to an immoveable object or be very heavy. 3) The power to the amplifier/filter system should be very well grounded and should be cleaned and stabilized since input power fluctuations are also amplified with the signals. 4) Scour must be eliminated below the detector so the particles move above and close to the coils and so that erosion and destruction of the detector is prevented. (Erosion of one 90 kg block occurred in 1981. The detector was transported approximately 30 m down stream.) 5) The detector should be protected from physical damage due to cobble impact with sheet aluminum. 6) Good protection for wires between the detector and amplifier/recorder is essential to maintain continuity during flood.

The Amplifier/Recorder

The amplifier/recorder system has also evolved in response to noise, signal, and counting constraints and provide further insight into the requirements for installation. From 1981 to 1986 amplification and recording were accomplished with commercially-available-flat-bed-multichannel recorders capable of sensing zero-to-one millivolt pulses. These devices provided paper output of good quality (Custer and others, 1987; Bunte, 1991), but had some practical problems. The paper speeds were necessarily high to gain enough separation between signals to allow counting (1-5 m/h and more). Such paper rates require someone to be present to change paper, add ink, fix jams and generally maintain the machine. Such paper rates also produce large physical volumes of output. Furthermore, someone was required to physically count the many signals produced by moving particles during an event. There were also electronic problems with the recorders. The signal to noise ratio was sometimes a problem. Noise filters and amplifiers improved the output. Early amplifier/filter devices were designed and built for the flat-bed recorders (Spieker and Ergenzinger, 1990). The more recent amplifier/recorders were designed for computer interfaced data acquisition by George Christaller and his students at the Institut für Sensor Technik in the Techniche Fachhochschule, Berlin, Germany.

> "The electronics consist of 4 main parts, a double auto zero amplifier (DAZA), an analog multiplexer (MUX), a data acquisition controller (DAC) with digital converter (ADC) and some auxiliary electronics with a micro-controller" (Ergenzinger and others, 1992, p. 5).

The amplifiers enhance the signal up to 10^{+6} times. The newest device also contains noise filters and the capability of visual output on the computer monitor. The computerization of the system allows automated signal counting which replaces the more tedious counting of signals from chart-recorder paper.

The Output

Temporal and Spatial Variability of Transport (Detector Output)

The detector system produces output which shows the moment coarse-magnetic-bed-load-particle motion begins (Figure 4) (Custer and others, 1987). The output also records the temporal and spatial variability of bed-load-particle (Figure 4). These results are not unique from the perspective of our growing understanding of coarse-grained bed-load-transport phenomena in flumes and a few natural streams (Gomez and others, 1989). However, the device does provide a unique means of electronically detecting natural transport initiation and bursts without elaborate, physically demanding, expensive, and problematic physical samplers or sediment traps.

Intuitively, one might expect a good temporal relationship between stream discharge and transport rate. However, examination of stage and particle-transport-signal counts plotted on a common time axis suggest our intuition is incorrect (Bunte and others, 1987; Custer and others, 1987; Bunte, 1991). The records (Figure 5) reveal that some hydrograph peaks have no associated coarse-particle-transport-count peaks (3, 9); some coarse-particle-transport-count peaks come on the falling limb of the hydrograph (2, 6, 7, 8 10, 11); some coarse-particle-transport-count peaks come at the same time as the hydrograph peaks (1, 4, 5, 6, 11); and some hydrograph events have multiple sediment transport peaks (6, 10, 11). Data sampled at five-minute intervals reveals even less correlation with discharge (Bunte, 1991; 1992). Analysis of the five-minute records show that the amplitudes of the bed-load pulses increase with increasing discharge while the frequency decreases until a periodicity of abut 1.5 hours was reached during peak flow (Bunte, 1991, p. 145). Examination of even more detailed computer-counted records of coarse-particle detection during high water on the fifth and sixth of June 1991 (Figure 6). On these dates, there was actually a coarse-particle-transport lull on most sensor channels at peak stage with a burst before peak and a larger burst after the peak. Furthermore, the count-rates at the start of measurement on the lower detector are higher than the count rates at the higher detector under hydraulic conditions which one would normally assume are identical since the two detectors are only 28 m apart and the measurements were taken simultaneously.

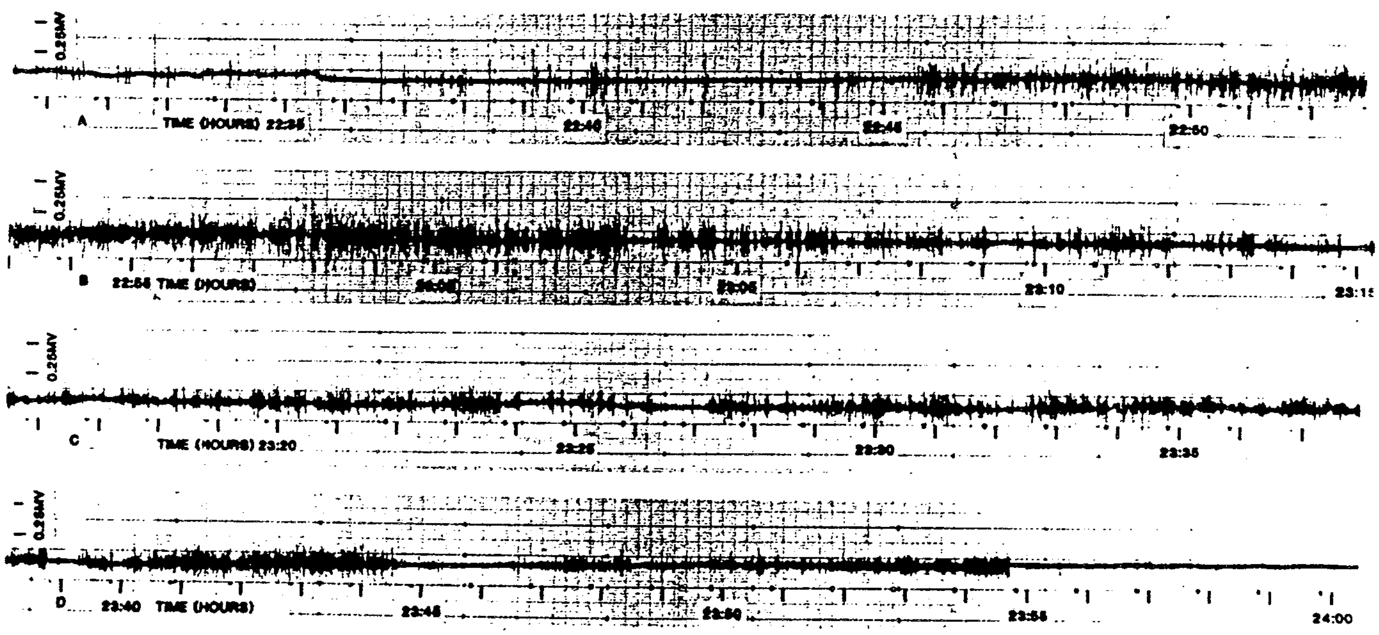


Figure 4. Chart-recorder record of potential versus time from a from a 1981-style detector on 2 May 1982 (from Custer and others, 1987, p. 24).

Detector data from 1991 show that transport fluctuates spatially as well as temporally at Squaw Creek (Figure 6) (Ergenzinger and others, 1992). Discharge on this date peaked at 7.2 m^3s^{-1} (0.4 m^3s^{-1} north channel; 6.8 m^3s^{-1} main channel). The gap from about 10:07 to about 13:35 is a period during which computer data acquisition briefly failed. Detectors in the upstream detector log (about 28 m up-stream of the road bridge) show sediment movement at different times and locations. Most of the transport occurs in the channel rather than on the shallower bar at the up-stream detector, but at the same time over the down-stream detector log (at the road bridge), transport tends to be focused in the center of the stream with motion both on the bar, the boundary between the bar and channel and in the channel.

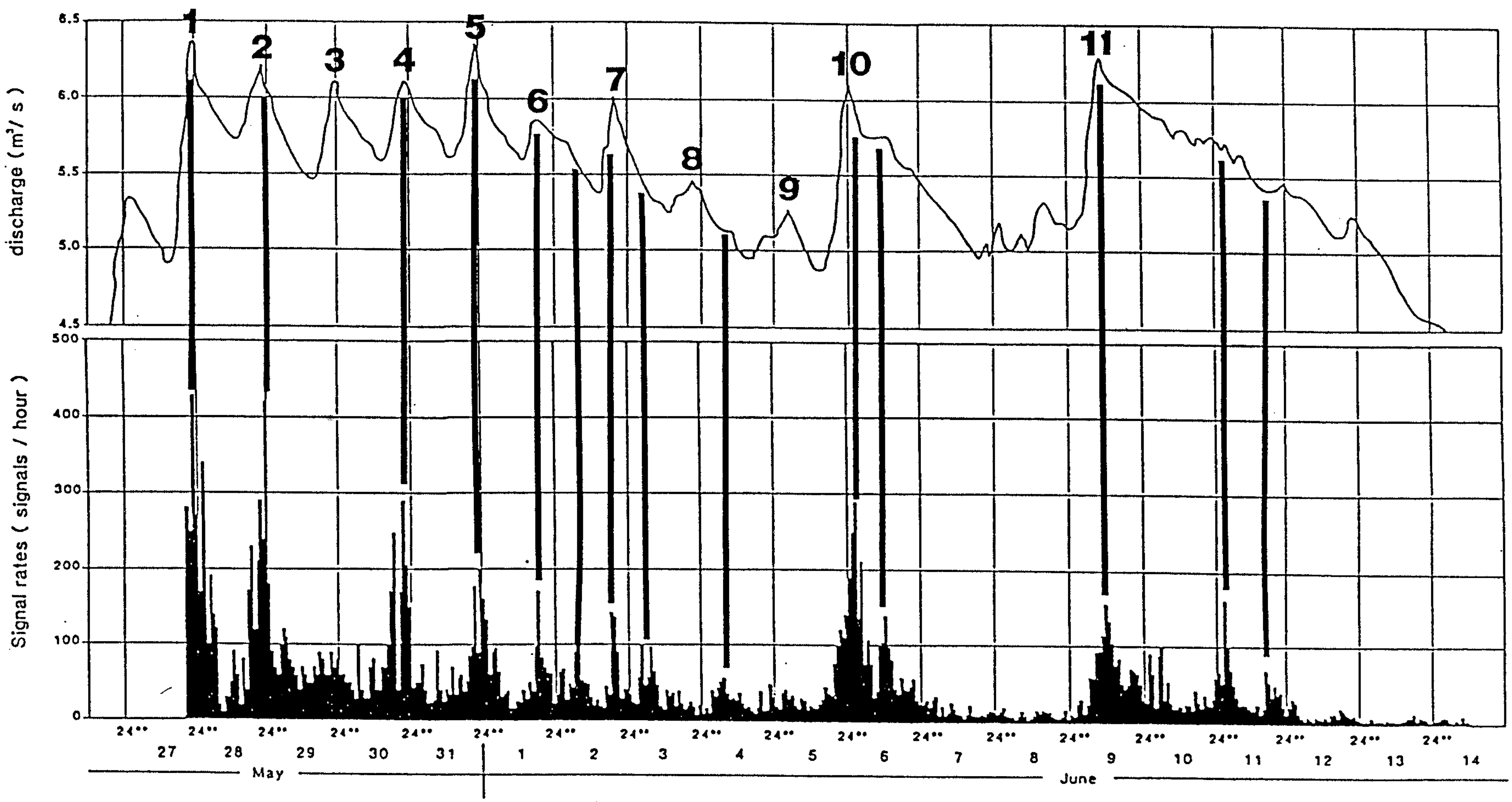


Figure 5. Discharge versus particle count for transport year 1986 (modified from Bunte, 1991, p. 98). Numbers refer to discharge peaks for reference in text. Bold vertical lines are included to help guide the eye between transport peaks and the hydrograph for comparison purposes.

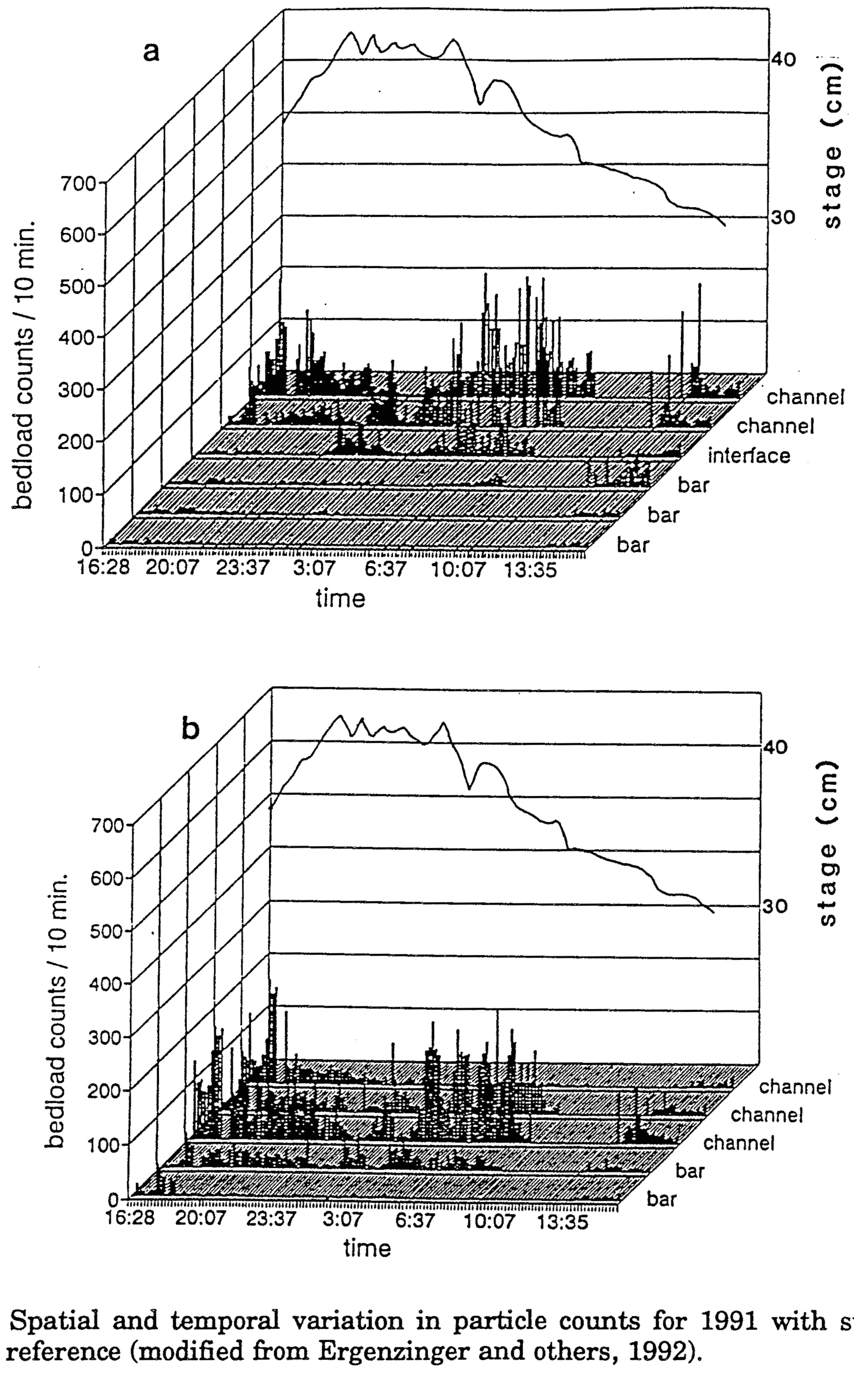


Figure 6. Spatial and temporal variation in particle counts for 1991 with stage plotted for reference (modified from Ergenzinger and others, 1992).

Discussion

Validation

To test whether physical bed-load-transport samples can reproduce transport patterns which were out of phase with respect to discharge peaks, bed load samples were taken in the channel at the road bridge on Squaw creek between 1.85 and 3.35 m from the right-bank bridge wall with a large bed-load-net sampler in 1988 (Bunte, 1990, 1992). (No direct calibration of the detector was possible because of electronic noise problems in a failing amplifier. A good example of the importance of eliminating noise and of the need for a good electronic instrument base and good electronic technical support.) The sampler opening was 1.6 m wide and 0.3 m high. The net was 3 m long and was composed of 10 mm mesh. The large mesh size was selected because the particles detected by the passive magnetic detector are pebble size (Ergenzinger and Custer, 1983), and because of sampler-plugging and sampler handling problems anticipated when the net was full if a smaller mesh size was used (Bunte, 1990). Although only a few samples could be taken because of the size and unwieldiness of the sampler, variable transport rates are again found, and the largest transport rate occurred on the receding limb of the hydrograph (Figure 7). (The line in figure 7 reflects discharge, the histogram reflects bed-load-transport rate.) Although transport rate does broadly increase with discharge as expected, at any given discharge even for a single event, the transport rate can vary by an order of magnitude. This variation is undoubtedly related to whether a transport pulse is passing at the moment of sampling rather than discharge-controlled transport. Similar results using other systems have been reviewed by Gomez and others (1989). The electronic count data from 1986 and the hand-sampled transport data from 1988, as well as work by others in flumes and on streams suggest that factors in addition to simple hydraulic thresholds which arise from discharge must control transport pulses.

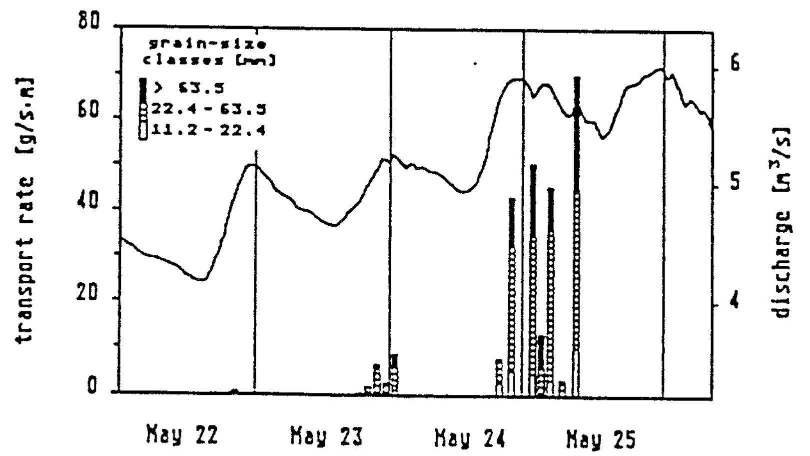


Figure 7. Results of physical bed load samples taken in 1988 with a large-net sampler (from Bunte, 1990, p. 226).

Factors Influencing Spatial and Temporal Variability.

The factors which influence the spatial and temporal variability of coarse-bed-load transport in a natural fluvial system such as that at Squaw Creek are probably both hydraulic and geomorphologic. Variability exists even under the highly controlled conditions of flume experiments (for example, Hubbell and others 1987; Iseya and Ikeda, 1987). The use of hydraulic factors to predict sediment transport have been the subject of intense scrutiny for many years (for example, Bagnold, 1977, Andrews, 1983; Carling 1983; Bathurst, 1987; Bathurst and others, 1987). Given the spatial and temporal variability and the lack of correlation between pulse and discharge or stage, broad generalizations which use stream-wide stream power (a function of discharge and slope) or even stream-wide shear stress (a function of slope and depth) appear unlikely to explain the observed behavior. Local spatial and temporal variation of stream power per unit bed width, local shear stress, or local shear velocity in response to local changes in velocity, depth, and slope at different locations on the stream bottom may better explain the pulses. However, instantaneous measurement of local changes in depth, slope, and velocity profiles for the water at many locations is very labor intensive and has rarely been accomplished in natural systems. Iseya and Ikeda (1987) reviewed factors which might result in such motion. Factors include hysteresis (Milhous and Klingeman, 1973; Griffiths, 1989; Kuhnle, 1992), armour development (Gomez, 1983), kinematic waves (Langbein and Leopold, 1968; Ried and others, 1985), longitudinal sorting processes (Iseya and Ikeda, 1987), and migrating bed forms (Hubbell and others 1985; Whiting and others, 1988; Kuhnle and Southard, 1988). The importance of many of these factors cannot be tested with the detector, but migrating bed forms might be visible in detector records from two detectors separated by a short distance. Visual examination of figure 6 does not reveal an identifiable pulse of sediment transport at the upper detector followed by a similar identifiable pulse at the lower detector.

Change in slope is an important hydraulic variable which is rarely measured in space and time at natural sites during sediment transport (Prestegaard, 1983). Some preliminary measurements of spatial-temporal change of slope at Squaw Creek have been made by laying hoses longitudinally along the stream bottom at several locations. The up-stream end of the hoses are bent so they face down stream to reduce the influence of flow velocity, and constitute the point of measurement for the head. At the downstream end, a clear tube is attached to the hose and is raised out of the stream. The difference in height between the water level in the tube and the level of water adjacent to the tube is the change in head. The head difference divided by the distance between the upstream end of the hose and the place where the clear tube is raised represents the slope. If multiple hoses are installed, the slope between the two hoses can be measured. This arrangement allows water-surface slope assessment through space and time at least to a limited extent along two banks. In 1986 and 1988 slope tubes were spaced at 17.3 m on the left bank (Bunte, 1991). Data from head in the two hoses revealed that slope varied through time as the flood progressed. In 1991, slope tubes were installed on both banks (Ergenzinger and others, 1992) (Figure 8). This data shows that water-surface slope varies between 0.022 and 0.018. Generally the water-surface slope on the right bank is higher than on the left bank but in the morning the two slopes cross and values are highest on the left bank for a while. The changes in water-surface slope were on the banks and so cannot be compared easily to the transport pulses in the stream channel itself, but the data suggest that slope varies measurably temporally and spatially. This result is not surprising since the gravel bar (and multiple channels) introduce spatial variation in bed-surface slope. The results reinforce the statement by Gomez and Church (1989, p. 1183),

> "...In view of this, strictly "local" hydraulic parameters should be utilized. The use of mean or total hydraulic values represents a channel-wide integration before the transport calculation, whereas strict observance of the form of the formulae requires that the transport be calculated first. Since the formulae are non linear, the effect may be important."

More work with collection and analysis of natural slope data at many points on the stream bed is needed to better understand relationships between depth, slope, shear stress, stream power, and coarse-bed-load-transport pulses. There may even be a relationship between the pulses and the water surface slope related to turbulent (unsteady) flow cells. Such interaction might be causative (cells develop and induce transport when the whole bed is near threshold) (Ergenzinger and others, 1992), responsive (water surface slope responds to sediment transport waves but does not drive them), interactive (water-surface slope and transport interact) or unrelated.

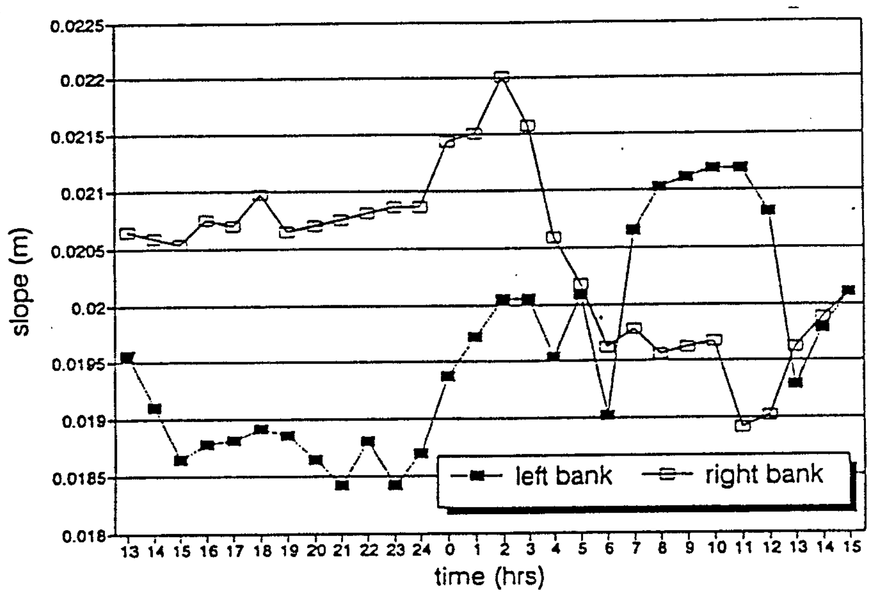


Figure 8. Slope variations near the right and left banks during 1991 (cf. Figure 6).

Although hydraulic factors are generally assumed to control sediment transport in gravel bed rivers, geomorphic factors also have an impact on natural sediment transport and supply (Bathurst, 1987; Bluck, 1987; Bugosh and Custer, 1989; Griffiths, 1989; Hassan and Ried, 1990; Hoey and Sutherland, 1991; Jong, 1991 and 1992; Naden and Brayshaw, 1987; Warburton, 1992). Bathurst (1987) suggests an interaction between sediment storage and armour layers may explain particle transport events which lag behind the peak discharge or even occur before the peak.

> "Break-up of this layer near the peak flow releases material from below the layer and allows greater transport during the falling limb of the hydrograph (eg. Klingeman and Emmett, 1982; Reid and others, 1985). The opposite may occur if, instead, the rising limb is able to tap supplies of sediment which have been accumulating along the channel since the previous flood" (Bathurst, 1987, p. 283.).

Bursting sediment transport at a smaller temporal scale than discharge events (Figure 4) may also be related to the creation and destruction of cluster bed-forms in gravel bed rivers (eg: Reid and Frostick, 1984; Hassan and Reid, 1990; Jong, 1991). Cluster organization and destruction might control entrainment thresholds (Brayshaw, 1985).

Other supply factors may influence the "erratic" sediment transport patterns. One explanation of the fact that there are more particle counts at the lower detector than the upper in figure 6 may be that bed material was stored between the two detectors during an earlier event and moved over the detector during the rising limb of the hydrograph during the period shown in the figure (Bunte, personal communication, 1992). Another explanation may relate to exposure of new material to threshold hydraulic conditions as stage increased and new bed features were flooded in or near the reach of interest. There are undoubtedly also interactions between channel features and the flow which may result in local changes in channel position, slope, and local flow direction or changes in channel or thalweg location which may occur within or up stream of the channel system under study (Griffiths, 1989; Naden and Brayshaw, 1987). For example, an alternative explanation for the higher count data at the onset of measurement in figure 6 is exposure of new sources in the channel north of the gravel bar (Figure 2) which are not available to influence the upstream detector block. Different channels are not required to cause supply fluctuations. As stage rises different parts of the channel are inundated and these areas contain different sized material. These newly inundated areas represent different supplies available for transport at different times. Other supply factors include simple bank erosion and collapse, disruption of the stream bottom by floating debris, and log-jam bursts (Bugosh and Custer, 1989). Sediment pulses may also be delivered by nearby tributary streams (Bathurst, 1987), debris flows or land slides near by, or be delivered upstream some distance at a much earlier time and simply arrive at the site many flood peaks later. Hydraulic conditions probably interact dynamically with supply in a complicated way to produce unsteady transport behavior.

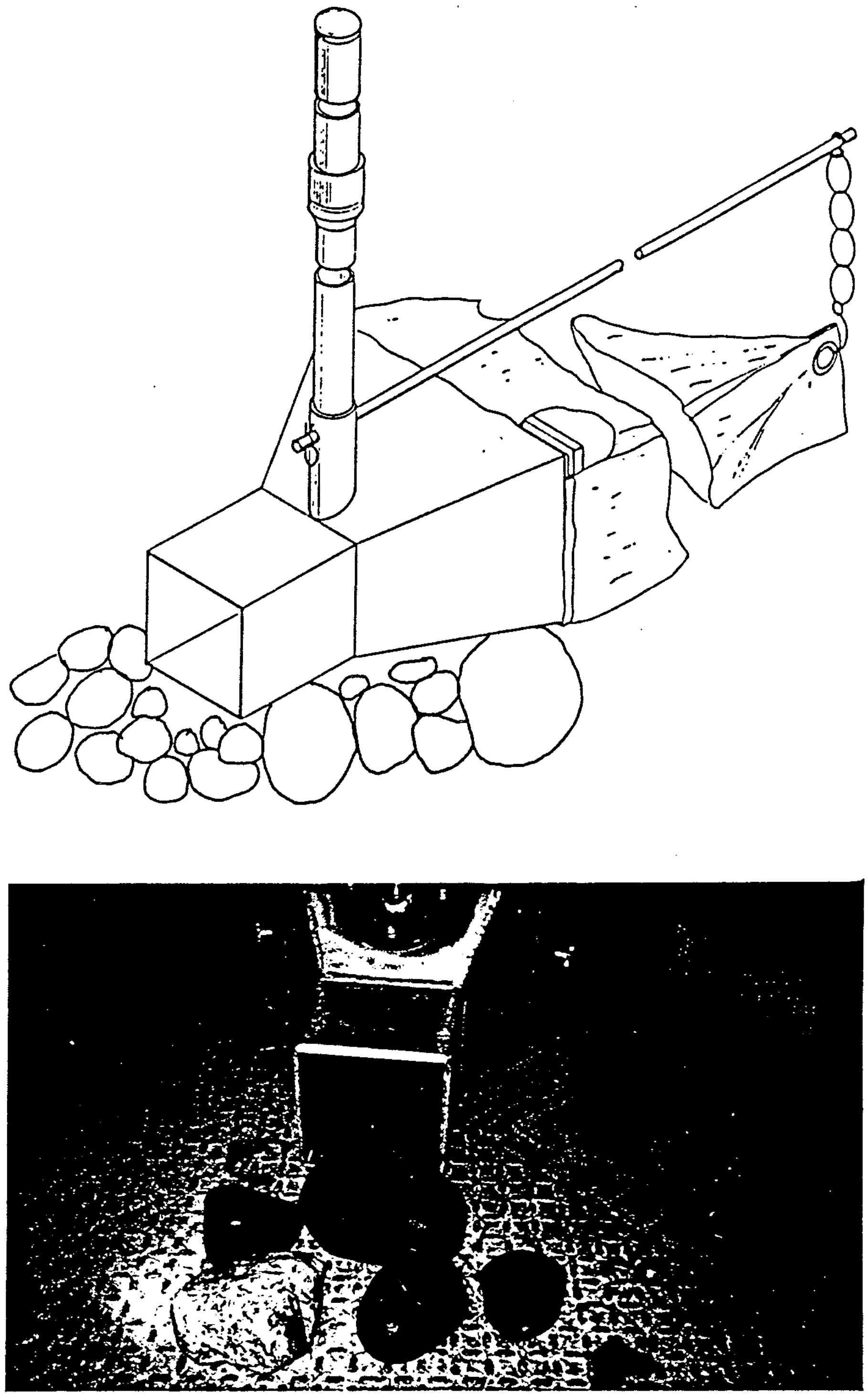
Calibration (Gravel-Transport Sampling Problems)

The detector output clearly shows spatial and temporal transport patterns, and permits analysis of transport with respect to discharge, but the data is displayed in counts-per-time units rather than the more conventional mass-per-time units (Both approaches include a per unit stream width term which varies depending on the detector, sampler width, and whether all width terms are summed for the stream.) Although the non-traditional units of counts (particles) per unit time are useful, there is interest in the relationship between mass transport and the number of particles moving. The history of calibration attempts provide insight both into the particle detector and into bed-load transport measurement problems in gravel-bed streams.

A simple proportionality approach was used in 1981 (Ergenzinger and Custer, 1983). The approach depends on knowledge of the proportion of detectable particles in the stream bed. Not all mobile coarse-bed-load particles are detected, only a portion of them are. A variety of factors influence detectibility. Factors include the amount of magnetic minerals in the rock composing the clast, the size of the clast, the orientation of the clast during passage over the detector, the height of the clast above the detector, and the velocity of the clast. The uncalibrated use of count data from the detector depends on the assumption that both detectable and nondetectible particles are moving past the detection point at any moment in time. Thus, there is a relationship between number of detectable coarse particles and the total number of particles passing the detector. Since no property of a detectable coarse particle is known to influence transportability in a way that is different from an undetectable coarse particle, the assumption of a relationship appears reasonable. Indeed, the presence of nondetectible particles is actually an advantage, because if all particles were detectable, the counters would probably be overwhelmed. Based on laboratory experiments with detectors used in 1981 only about 15% of the material larger than pebble size was counted and only about 5 % of all material in transport was counted (Ergenzinger and Custer, 1983). The ratio has probably changed as the detectors, filters, and amplifiers have changed, but generally the assumption is made that there is a proportional relationship between detectable and undetectable particles. To define the relationship of proportionality gravel-sized bed-load transport sampling is needed.

While there are several problems with the simple proportionality approximation outlined above, field sampling is no more straight forward (Bathurst, 1987; Church and others, 1987). The problems exist for other workers as well (for example, Gomez and Emmett, 1991) and are simply reiterated by calibration attempts at Squaw Creek. Given the nature of the count data, any calibration must be accomplished at the same location as the detection and must occur over the same time interval. Samples taken at different locations or different times cannot be used because of the temporal and spatial variability of transport demonstrated by the detector. Early attempts used a modified Muhloffer sampler (Ergenzinger and Custer, 1983; see Figure 3 B). The force of the water physically prevented the sampler from being moved forward into sampling position even with substantial levers. A 7.6 cm Helley-Smith sampler attached to a "fish" was tried, but cross currents prevented accurate positioning of the sampler over the detector. Hand-held Helley-Smith samplers from a foot bridge were also tried. Three problems resulted in abandonment of this technique. The first problem is bed contact. Physical sampling must occur at the down-stream detector, in the channel where the largest particles occur, at the point where most detection is observed. Here, there are cobbles and boulders nearly flush with the detector. The hand-held sampler rests in irregular contact on these boulders. This poor bed contact means sediment bypasses the sampler because the nozzle is held above the transport site around the boulder (Figure 9 a). A second problem has to do with orifice size. The photograph of particles trapped in the large-net sampler next to the 15 cm (6 inch) Helley-Smith sampler (Figure 9 b) graphically illustrates the need for a large sampler orifice. Clearly an orifice larger than six inches is needed, but even the 15 cm Helley Smith sampler with a 2.5 m handle levered against the foot bridge has presented too much surface area to the stream to be successfully held in a known position safely. A final problem is the electronic noise induced by the metallic sampler. The number of counts before and after sampling can be determined, but during sampling, noise makes counting impossible. Given the temporal variability of transport, the time of sampling and counting must coincide. Thus even if a sample is obtained, one can not be certain it is representative of material in transport during the counting interval.

Because of the difficulties with hand-held samplers, considerable discussion has taken place regarding Birkbeck (Lewis, 1991) and vortex samplers (Klingeman and Milhous, 1970; Tacconi and Billi, 1987). For Birkbeck samplers, there have been concerns about filling and emptying rates. In 1991, an inexpensive large-net sampler was further modified in order to over come the problems previously encountered with the small hand-held samplers (Bunte, 1992). However, the 3 x 1.6 x 0.3 m net was filled to capacity in 5 minutes and was too heavy to physically be removed from the stream until low flow. Emptying a Birkbeck type sampler multiple times during a transport event has been daunting, as have the problems of changing sampler efficiency during filling, particularly as sediment fill approaches the top of the sampler. A small, vortex sampler has been constructed for part of the channel width just below the downstream detector log but has not yet been in the stream during a transport event. This sampler also has volume and mass problems with output during periods of high transport rate, and also has some problems with sampler efficiency for particles smaller than 10 mm (Bathurst, 1987). The problems of physically sampling a gravel bed river with abundant cobbles and boulders is a significant problem which requires careful attention to sampling time, sampling



A

B

Figure 9. A) Bed-contact problem with Helley-Smith sampler. B) Particle-transport size trapped in the large-net sampler as compared to a 15 cm Helley-Smith orifice.

duration, sampler position, sampler efficiency, bed contact, net size, and how to lift large samples from the stream without interfering with the counting detector.

Applications

Although correlation of detector counts to mass transport beyond the simple proportional approximation has proven difficult, the detectors produce excellent data for appropriate applications. Successful application depends upon experimental design and the question posed. Applications exist both for model verification and for monitoring.

If the model question is, "Do modeled predictions of the initiation of gravel transport in natural rivers match those observed," the detector can monitor threshold of initiation of gravel transport and provides a comparison. (The assumption is that both detectable and nondetectible particles begin to move at the same time. This seems to be a reasonable inference since rock magnetism is not known to inhibit or enhance particle motion.) Thus modeled conditions for initiation of gravel transport can be tested in natural settings because the detector records continuously and precisely detects the initiation of transport.

If the model question is, "Where does bed-load transport take place in the stream bottom?", the detector can provide this information and show how the particle-transport location changes through time and in response to changing hydraulic variables. Figure 6 shows that gravel particles do not move everywhere on the stream bottom, that the position of motion is different at different longitudinal stream positions 28 m apart, and that the position changes through time. Such information should be help with development of predictive models for bed-load transport and channel change in gravel-bed streams.

If the model question is, "Under what hydraulic conditions can gravel transport be expected to begin?" the detector can provide real-time data regarding the instant motion begins if continuous monitoring of hydraulic variables exists. Knowledge regarding threshold of motion requires measurements be taken just as motion starts. Identification of the moment gravel transport begins in a natural stream is very difficult with standard samplers. The detector is far superior to the technique of asking many personnel to stand in the river sampling until the moment of onset of gravel transport is discovered. Thus, the detector's usefulness extends past model testing to monitoring purposes for this question.

Such detection also has practical applications. If the environmental monitoring question is, "Have pebbles begun to move on the bed at this engineering structure?", the detector can provide real-time data. For example, if a detector were attached to a bridge pier, initiation of erosion should be detectable for monitoring or in extreme cases for disaster assessment with or without models. Applications might include particle motion warning systems for small hydroelectric facilities or early warning systems for onset of erosion of engineered facilities in streams. Thus the detector might be used for disaster preparedness as well as for development of sound engineering practice in gravel-bed streams.

Applicable Sites

The detector will work in many (but not all) gravel-bed streams. The primary requirement is the presence of naturally magnetic material larger than 2 cm in the stream bed. Such material is more widely available than one might expect. Squaw Creek drains an Eocene andesitic volcanic complex in southwestern Montana. Thus, any andesitic volcanic terrain such as those associated with subduction in the Cascade Range and Coast Range of Western North America, the Aleutian and Alaska Range in Alaska, the Andes of South America, and mountains in the Philippines, New Zealand, and Japan is suitable for the application of the magnetic tracer technique for pebbles and cobbles in stream beds. Basaltic terrains are also good candidates because of the magnetite content of basalts (eg: Snake River Plain, Columbia Plateau, Iceland, Deccan Plateau of India). Other metamorphic and intrusive terrains may also contain pebbles sufficiently magnetic to be detectable. Indeed, the idea for detection of naturally magnetic material arose from the Archean Stillwater Complex in Montana which is associated with banded iron formation. Although that terrain proved to be so magnetic that there was fear of saturation of the detectors with signals, other areas of banded iron formation such as those in Minnesota and Australia have potential. Even clastic sedimentary terrain may contain sufficient magnetite to work to produce detectable clasts. The Virgelle Sandstone in Montana has magnetite placer beds that produce magnetic clasts. Immature sediments derived from source terrains with magnetite may also produce clasts that are detectable. The primary point is that there are many regions in the world where detection is possible. The best way to determine whether the detector will work is to use a stud finder such as those used in the mineral industry to identify magnetic ore (about $5 in 1992). If pebbles and cobbles in a stream attract a stud finder, the site will probably prove suitable for the passive magnetic-particle detector. There are many such areas, particularly in mountainous regions of the world where the technique should work.

Conclusions

1) There is a coarse bed-load transport detector available which employs electronic signals induced by naturally-magnetic particles larger than gravel size. The natural tracers do not have to be placed in the stream artificially. The detector has been improved not only by reduction of the size of the components, but also with the addition of filters, amplifiers, and computer-counting circuits. The improvements have increased the sensitivity of the detector. The attached computer circuit

automates the counting and allows easy data transfer to spread sheets or other computer programs.

2) The detector can be successfully operated at sites with naturally magnetic clasts. Such clasts should be expected in many areas where gravel-bed streams exist (Cascade Range, Aleutian Range, Andes, Japan, Iceland, New Zealand.

3) Results with the detector show that threshold of gravel motion can be detected in real time. The detector further shows that particle motion is temporally and spatially variable. The variability occurs at several scales. There is only a crude relationship between discharge peaks and stream-width-averaged peaks in the particle-counts data.

4) The explanation for the spatial and temporal variability is uncertain but may be related to complex interactions between very "local" (meter scale) hydraulic conditions (slope, shear stress, water depth, mesoscale flow cells) and geomorphic conditions that relate to the particle interactions, stream-bed character and sediment supply factors.

5) Calibration of the detector signals has proved to be problematic and needs work. The problems are similar to those reported by others working in coarse gravel bed rivers. Even without calibration there may be applications for the detector including real-time detection of the threshold of transport initiation and identification of the location of transport on the stream bottom.

6) Future research with the instrument needs to focus on spatially and temporally continuous measurements of hydraulic variables such as water-surface slope, velocity, depth, and shear stress as well the calibration of the detector signals with bed-load samples with a more appropriate sampling devices. New approaches are needed to assess the issue of sediment supply and the dynamic interaction between hydraulic and geomorphic factors which produce the variable transport rates.

Acknowledgements

Much of the effort and the ideas reviewed here have been drawn from the work of others. Significant contributions have been made by Peter Ergenzinger and Kristin Bunte, George Christaller, Carmen de Jong, Raymond Spieker, Nick Bugosh, Brent Anderson. Paul Carling, Jon Williams, Mark Glaister, and Harriet Orr contributed perspective and collected data on velocity distribution, turbulence, and hydrophone data in 1991 which is not reported here. Analysis of their data should be forthcoming soon. During the early years of this research Castle Rock Baptist Bible Camp provided power and shelter. Big Timber Works Incorporated, Hogin Machine and Welding, and many others too numerous to list assisted with construction, late-night measurement, and good cheer. Much credit should go to these individuals. Responsibility for any errors or omissions remain with the author.

Over the past 10 years this work has been supported financially by the Deutsche Forschungsgemeinschaft, the Freien Universität Berlin, and the Technische Fachhochschule Berlin, in Germany; The European Office of the U.S. Army Corps of Engineers; the Institute for Freshwater Ecology, and the Institute of Oceanographic Sciences in the United Kingdom; the U.S. Army Corps of Engineers Water Ways Experiment Station, and by Montana State University in the United States.

References Cited

Ackers, P., and Thompson, G., 1987, Reservoir sedimentation and influence of flushing, in Thorne, C.R., Bathurst, J.C., and Hey, R.D., (eds.), Sediment transport in gravel-bed rivers: New York, John Wiley and Sons. p. 845-868.

Andrews, E.D., 1983, Entrainment of gravel from naturally sorted river bed material: Geological Society of America Bulletin, v. 94, p. 1225-31.

Bagnold, R.A., 1977, Bedload transport by natural rivers: Water Resources Research, v. 13, p. 303-312.

Bathurst, J.C., 1987, Measuring and modelling bedload transport in channels with coarse bed materials, in Richards, K., (ed.), River channels environment and process: Basil Blackwell, Oxford, United Kingdom, p. 272-294.

Bathurst, J. C., Graf, W.H., and Cao, H.H., 1987, Bed load discharge equations for steep mountain rivers, in Thorne, C.R., Bathurst, J.C., and Hey, R.D., (eds.) Sediment transport in gravel-bed rivers: Chichester, John Wiley and Sons, p. 453-491.

Bluck, B.J., 1987, Bed forms and clast size changes in gravel-bed rivers, in Richards, K. (ed.), River channels environment and process: Basil Blackwell, Oxford, United Kingdom, p. 159-178.

Brayshaw, 1985, Bed microtopography and entrainment thresholds in gravel bed rivers: Geological Society of America Bulletin, v. 96, p. 218-223.

Bugosh, N., 1988, Field verification of predictive bedload formulas in a coarse-bedload mountain stream: Master of Science Thesis, Montana State University, Bozeman, MT, 130 pp.

Bugosh, N. and Custer, S.G., 1989, The effect of a log-jam burst on bedload transport and channel characteristics in a headwaters stream, in Woessner, W.W., and Potts, D.F., (eds.), Proceedings of a symposium on headwater hydrology in June 1989: American Water Resources Association Technical Publication Series, TPS 89-1, p. 203-211.

Bunte, K., Custer, S.G., Ergenzinger, P., and Spieker, R., 1987, Messung des grobgeschiebetransportes mit der magnettracertechnik. Deutsche Gewaesserkundliche Mitteilungen, v. 31, pp. 60-67.

Bunte, K., and Ergenzinger, P., 1989, New tracer techniques for particles in gravel bed rivers. Bulletin de la Socie'te Ge'ographique de Lie'ge, v. 25, p. 85-90.

Bunte, K., 1990, Experiences and results from using a big-frame bed load sampler for coarse material bed load, in Proceedings of two Lusanne Symposia, August 1990: IAHS publication number 193, p. 223-230.

Bunte, K., 1991, Untersuchung der zeitlichen variation des grobgeschiebe-transportes und seiner korngroessenzusammensetzung (Squaw Creek Montana, USA): Ph.D. Thesis, Freien Universität, Berlin, Germany, 223 p.

Bunte, K., 1992, Particle number grain-size composition of bedload in a mountain stream, <u>in</u>, Billi, P., Hey, R.D., Thorne, C.R., and Tacconi, P., Dynamics of gravel-bed rivers: New York, John Wiley and Sons, p. 55-68.

Carling, P.A., 1983, Threshold of coarse sediment transport in broad and narrow natural streams: Earth Surface Processes and Landforms, v. 8, p. 1-18.

Carling, P.A., 1987, Bed stability in gravel streams, with reference to stream regulation and ecology, <u>in</u> Richards, K. (ed.), River channels environment and process: Basil Blackwell, Oxford, United Kingdom, p. 327-347.

Church, M.A., McLean, D.G., and Walcott, J.F., 1987, Bed load sampling and anaylsis, <u>in</u> Thorne, C.R., Bathurst, J.C., and Hey, R.D. (eds.), Sediment transport in gravel bed rivers: John Wiley and Sons, New York, New York, p. 43-88.

Custer, S. G., Bugosh, N., Ergenzinger, P.E., Anderson, B.C., 1987, Electromagnetic detection of pebble transport in streams: A method for measurement of sediment transport waves, <u>in</u> Ethridge, F. and Flores, R., (eds.), Recent developments in fluvial sedimentology: Society of Economic Paleontologists and Mineralogists Special Publication 39, pp. 21-26.

Ergenzinger, P., and Conrady, J., 1982, A new tracer technique for measuring bedload in natural channels: Catena, v. 9, p. 77-80.

Ergenzinger, P.E., and Custer, S.G., 1983, Determination of bedload transport using naturally magnetic tracers: First experiences at Squaw Creek, Gallatin County, Montana: Water Resources Research, v. 19, p. 187-193.

Ergenzinger, P., and Schmidt, K.H., 1990, Stochastic elements of bed load transport in a step pool mountain River, in Hydrology in mountainous regions II -- artificial reservoirs, Proceedings of two Lussane Symposia: IAHS Publication Number 194, p. 39-46.

Ergenzinger, P., Christaller, G., and Jong, C. De, 1992, Interrelationships between bedload transfer and river bed adjustment in Mountain Rivers: An example from Squaw Creek, Montana, in Theoretical geomorphology (Proceedings of the Leeds meeting of the British Geomporphologic Research Group (BGRG)). In review.

Gomez, B., 1983, Temporal variations in bedload transport rates: the effect of progressive bed armouring: Earth Surface Processes and Landforms, v. 8, p. 41-54.

Gomez, B., Naff, R.L., and Hubbell, D.W., 1989, Temporal variations in bed-load transport rates associated with the migration of bedforms: Earth Surface Processes and Landforms, v. 14, pp. 135-156.

Gomez, B., and Church, M., 1989, An assessment of bed load sediment transport formulae for gravel bed rivers: Water Resources Research, v. 25, p. 1161-1186.

Gomez, B., and Emmett, W.W., 1991, Comments on sampling bedload in small streams, in Fifth Federal interagency sedimentation conference proceedings: Washington, D.C., Federal Energy Regulatory Commission, p. 2-65--2-72.

Griffiths, G.A., 1989, Conversion of braided gravel-bed rivers to single-thread channels of equivalent transport capacity: Journal of Hydrology (New Zealand), v. 28, p. 63-75.

Hassan, M.A., and Reid, I., 1990, The influence of microform bed roughness elements on flow and sediment transport in gravel bed rivers: Earth Surface Processes and Landforms, v. 15, p. 739-750.

Hoey, T.B,., and Sutherland, A.J., 1991, Channel morphology and bedload pulses in braided rivers: a laboratory study: Earth Surface Processes and Landforms, v. 16, p. 447-462.

Hubbell, D.W., Stevens, H.H., Skinner, J.V., and Beverage, J.P., 1985, New approaches to calibrating bedload samplers: Journal of the Hydraulics Division of the American Society of Civil Engineers, v. 111, p. 677-694.

Hubbell, D.W., Stevens, H.H., Skinner, J.V., and Beverage, J.P., 1987, Laboratory data on coarse-sediment transport for bedload-sampler calibrations: U.S. Geological Survey Water-Supply Paper 2299, 31 p.

Iseya, F. and Ikeda, H., 1987, Pulsations in bedload transport rates induced by a longitudinal sediment sorting: A flume study using sand and gravel mixtures: Geografiska Annaler, v. 69 A, p. 15-27.

Jong, C. De, 1991, A reappraisal of the significance of obstacle clasts in cluster bedform dispersal: Earth Surface Processes and Landforms, v. 16, p. 737-44.

Jong, C. De , and Ergenzinger, P. 1992, Unsteady flow, bedload transport and bed roughness responses in steep mountain torrents, in Proceedings of the fifth international symposium on river sedimentation and sediment management, Karlsruhe, 1992: ISRS, v. 1, p. 185-192.

Jong, C. De, 1992, Thresholds for channel change on two contrasting pro-glacial river fans, West Greenland: Geografiska Annaler, v. 74 A, p. 1-12.

Klingeman, P.C., and Milhous, R.T., 1970, Oak Creek vortex bed-load sampler, in Seventeenth annual Pacific Northwest meeting of the American Geophysical Union proceedings: Washington, D.C., American Geophysical Union, 17 pp.

Kuhnle, R.A., 1992, Bed load transport during rising and falling stages on two small streams: Earth Surface Processes and Landforms, v. 17, p. 191-197.

Kuhnle, R.A., and Southard, J.B., 1988, Bed load transport fluctuations in a gravel-bed laboratory channel: Water Resources Research, v. 24, p. 247-260.

Lagasse, P.F., Schall, J.D., Johnson, F., Richardson, E.V., Richardson, J.R., and Chang, F., 1991, Stream stability at highway structures: U.S. Department of Transportation Federal Highway Administration Hydraulic Engineering Circular 20, 195 p.

Langbein, W.B., and Leopold, L.B., 1968, River channel bars and dunes-theory of kinematic waves: U.S. Geological Survey Professional Paper 422-L, 20 p.

Lewis, J., 1991, An improved bedload sampler, in Fifth Federal interagency sedimentation conference proceedings: Washington, D.C., Federal Energy Regulatory Commission, p. 6-1 - 6-15.

Milhous, R.T., and Klingeman, P.C., 1973, Sediment transport system in a gravel-bottomed stream, in Hydraulic Engineering and the Environment, Proceedings of the 21st Annual Hydraulics Division Special Conference in Bozeman, Montana: American Society of Civil Engineers, p. 293-303.

McMannis, W.J., and Chadwick, R.A., 1964, Geology of the Garnet Mountain Quadrangle, Gallatin County, Montana: Montana Bureau of Mines and Geology Bulletin 43, 47 p.

Naden, P.S., and Brayshaw, A.C., 1987, Small- and medium-scale bed forms in gravel bed rivers, in Richards, K. (ed.), River channels environment and process: Basil Blackwell, Oxford, United Kingdom, p. 249-271.

Neill, C.R., and Hey, R.D., 1982, Gravel-bed rivers: Engineering Problems, in Hey, R.D., Bathurst, J.C., and Thorne, C.R., Gravel-bed rivers: Wiley, New York, New York, p. 15-25.

Prestegaard, K. L., 1983, Variables influencing water-surface slopes in gravel-bed streams at bankfull stage: Geological Society of America Bulletin, v. 94, p. 673-678.

Reid, I., and Frostick, L.E., 1984, Particle interaction and its effect on the thresholds of initial and final bedload motion in coarse alluvial channels, in, Koster, E.H., and Steel, R.J., (eds.), Sedimentology of gravels and conglomerates: Canadian Society of Petroleum Geologists, Memoir 10, pp. 61-68.

Reid, I, Frostick, L.E., and Layman, J., 1985, the incidence and nature of bedload transport during flood flows in coarse-grained alluvial channels: Earth Surface Processes and Landforms, v. 10, p. 33-34.

Richardson, E.V., Harrison, L.J., and Davis, S.R., 1991, Evaluating scour at bridges: U.S. Department of Transportation Federal Highway Administration Hydraulic Engineering Circular 18, 191 p.

Spieker, R., and Ergenzinger, P., 1990, New developments in measuring bed load by the magnetic tracer technique, in Erosion, transport and deposition processes. Proceedings of the Jerusalem Workshop, March-April 1987: IAHS Publication number 189, p. 169-178.

Tacconi, P., and Billi, P., 1987, Bed load transport measurements by the vortex-tube trap on Virginio Creek, Italy, in Thorne, C.R., Bathurst, J.C., and Hey, R.D., Sediment Transport in gravel-bed rivers: John Wiley and Sons, New York, New York, p. 853-616.

Wang, B., Bredthauer, S.R., and Marchegiani, E.A., 1987, Design problems in gravel-bed rivers, Alaska, in Thorne, C.R., Bathurst, J.C., and Hey, R.D., (eds.), Sediment transport in gravel bed rivers: New York, John Wiley and Sons, p. 869-96.

Warburton, J., 1992, Observations of bed load transport and channel bed changes in a proglacial mountain stream: Arctic and Alpine Research, v. 24 no. 3, p. 195-203.

Whiting, P.J., Dietrich, W.E., Leopold, L.B., Drake, T.G., and Shreve, R.L., 1988, Bedload sheets in heterogeneous sediment: Geology, v. 16, p. 105-108.

MONITORING GRAVEL MOVEMENT IN RIVERS

E. F. CHACHO, JR.

USA Cold Regions Research and Engineering Laboratory,
Fairbanks, Alaska, USA 99703-7680

R. L. BURROWS

U.S. Geological Survey, Fairbanks, Alaska, USA 99775-5170

W. W. EMMETT

U.S. Geological Survey, Lakewood, Colorado, USA 80225-0046

Radio transmitters were implanted in natural river gravel to locate and track the movement of coarse sediment (39 mm or larger) through a natural river reach. An automatic data acquisition system was developed to continuously monitor the radio-implanted sediment particles to determine the travel time of the rocks through a 362-m study reach. A total of 24 radio-tagged rocks was monitored either continuously or by periodic location surveys. The travel time of the rocks through the study reach is better related to specific gravity than weight of the particles. In addition the automatic data acquisition system continuously monitors the periods of motion and rest of natural river gravel implanted with radio transmitters equipped with motion sensors. The capabilities of the system are demonstrated by describing the motion and rest periods of a single rock for a two-month period including a number of flood events.

INTRODUCTION

The use of radio transmitters, implanted in individual sediment particles, to track the movement of coarse sediment in natural river systems was developed independently and simultaneously by Chacho, Burrows and Emmett (1989) and Ergenzinger, Schmidt and Busskamp (1989). Both groups have reported results from the application of the new technique: Emmett, Burrows and Chacho (1992) on seasonal travel distance; and Ergenzinger and Schmidt (1990) on the travel characteristics of cobbles in a step-pool river. In this paper, we describe the application of the technique and present examples of the type of data that can be acquired by continuously monitoring the motion and rest periods of various-size sediment particles and their travel time through a 362-m study reach.

The radio transmitter, enclosed in a hermetically sealed cylinder including battery and internal antenna, transmits a pulse at a specified frequency and period. Transmitter life is dependent on transmitting interval and battery size; for example, a 15- X 39-mm unit, transmitting at a pulse interval of about 0.5 msec, has a life of about 60 days, while an 18- X 72-mm unit, transmitting at the same pulse rate, lasts about 10 months. Natural sediment particles are collected from a study site and holes are drilled into the rocks to allow the implantation of the transmitters. The holes are sealed with epoxy and the rock is painted to aid in recovery at the end of the field season. Transmitters may be refurbished and used again. The tracking application of the technique requires the use of a directional antenna, radio receiver and signal strength meter.

In the continuous monitoring application of the technique, the transmitters are equipped with a motion-sensor which emits a signal at a single frequency but at a period which changes depending on whether the particle is at rest or in motion. A data acquisition system, utilizing a stationary antenna, radio receiver, automatic frequency scanner, digital processor and data logger is employed to continuously monitor the motion and rest periods of the motion-sensor equipped rocks.

The initial application of the data acquisition system took place in 1990, on Lignite Creek, in central Alaska. In this study, motion sensor-equipped radio transmitters, with unique frequencies, were installed in 17 sediment particles which were monitored by the data acquisition system and are referred to in this paper as the monitored rocks. An additional seven sediment particles were radio-tagged for periodic tracking only and are referred to as the unmonitored rocks. All 24 radio-tagged rocks were placed on a riffle at the head of the study reach. Periodically, following each flood event, all the rocks were tracked and travel distances measured.

STUDY SITE

Lignite Creek flows out of the foothills north of the Alaska Range into the Nenana River near Healy, Alaska. The drainage area of the basin is approximately 125 km^2. The lithologies of the basin are prone to mass wasting, producing high sediment loads on Lignite Creek. Average annual water discharge is 0.89 $m^3\ s^{-1}$. Flow during the study period in 1990 ranged from 0.5 to 16.3 $m^3\ s^{-1}$. Bedload transport, as computed from samples collected with a Helley-Smith bedload sampler, ranged from about 58 t day^{-1} to 365 t day^{-1}.

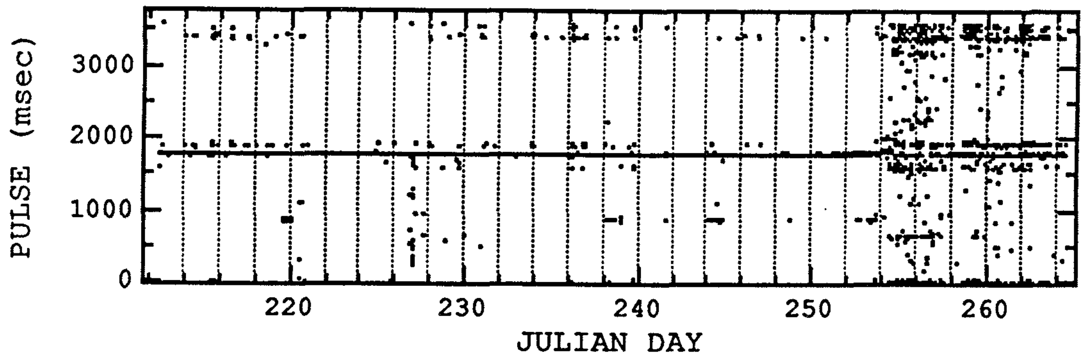
The study reach is 362 m in a straight line measure, from the point of introduction of the radio-tagged rocks to the bridge where the data acquisition system was located. The reach is relatively steep with slope changing from 0.0015 m m^{-1} upstream to 0.0008 m m^{-1} downstream. The reach is nearly straight, bending slightly from right to left downstream, and, depending upon discharge, may flow in more than one channel. Bed material varies from sand to boulders and is highly mobile; bedload transport occurs at all but the lowest flows.

INSTRUMENTATION

The data acquisition system sequentially scans a series of programmed radio frequencies. In this study 18 frequencies were scanned, 17 radio-tagged rocks and a stationary fixed pulse transmitter used to distinguish the beginning of a scan sequence. The system requires 15 sec to properly interrogate a given frequency. Therefore, in this study, each radio-tagged rock was interrogated once every 4.5 min. The motion sensor incorporates a delay time, that is, the time interval that the transmitter broadcasts the motion-indicator pulse rate following the occurrence of movement. In this specific application, the delay time was specified as 4.5 min so that each movement of a monitored rock during the total scan sequence would be recorded. This means that any movement of a monitored rock, even if it rolled only a partial revolution, would result in motion being recorded on the subsequent interrogation of that radio frequency in the scan sequence. If the rock started rolling and continued to move or moved in a series of short start and stop motions, multiple motion periods would be recorded on subsequent scans of that rock's frequency. Therefore, in this study, a motion or rest period is defined as movement or lack of movement in a 4.5 min time interval. Higher resolution of the motion periods is possible by reducing the number of radio frequencies to be scanned by an individual receiver.

The data record of motion and rest periods (4.5 min intervals) for a single radio-tagged rock is shown in Figure 1. For this specific transmitter, marks at approximately 1800 msec indicate periods of rest, and marks at approximately 800 msec indicate periods of motion. The upper plot is the original, raw data including radio interference and the record after the rock exited the study reach. Interference from extraneous communication signals is briefly discussed below. The break in record and the reduced quality of data as the rock was transported through the road bridge and out of the study reach is clearly seen beginning at day 253.714. The lower plot is the filtered data in which the extraneous interference and the record after the rock had exited the study reach have been removed. Four episodes of movement, which included multiple motion periods, were recorded near days 219, 238, 244 and 253. Two single motion periods were recorded on days 241 and 248.

Due to a number of high strength transmitters (i.e., railroad communication, television and radio repeaters) in the vicinity of the study site which operated near the frequencies used in this study (163-164 MHz), interference was a problem. In some cases the interference was minor and could be filtered out of the record (e.g., Fig. 1); in other cases the interference was so great that the entire



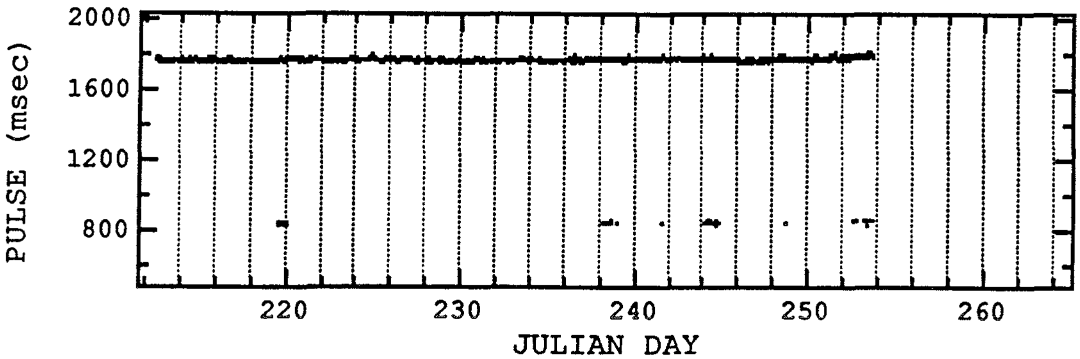


Figure 1. Data record of motion and rest periods for a motion sensor equipped radio-tagged rock (164.301). Upper plot is the original raw data, lower plot is the filtered data.

record of motion events for an individual rock had to be discarded. The interference problem has been greatly reduced in a subsequent study by using transmitter frequencies (148 MHz) much lower than the usual communication frequencies.

RESULTS

Due to the somewhat unique setting of this study site, where the location of the bridge over the stream caused a perturbation in the record as the monitored rock was transported under the bridge, two types of data sets can be recovered. The first is the record of the occurrence of the motion and rest periods, a time record which does not include any information on travel distance. The second set is the record of the time the monitored rocks exited the study reach, that is, the time required for transport over a known distance. In this paper we include examples and initial results from both types of data sets. Complete analysis and interpretation is beyond the scope and length limitations of this paper and will be reported in the future.

The data from the 17 monitored rocks was filtered to remove interference and analyzed to determine the occurrence and timing of motion events. Due to the interference described above, only one rock had a complete record of motion events for its entire transit time through the study reach. The complete set of motion events for this rock is described below.

A reach exit time was discernible for all 17 of the monitored rocks. In addition the reach exit time for the seven unmonitored rocks was estimated from the periodic location surveys. An initial analysis of these data sets is also presented.

MOTION EVENTS

The rock with a near-perfect record of motion events, radio frequency 164.301, was about mid-range in size and weight, and near the upper bound in specific gravity and reach exit date of the 24 radio-tagged rocks (Table 1). The discharge hydrograph and the motion events of this rock during its transport through the study reach is shown in Figure 2. As was shown in Figure 1, four flood events produced motion events with multiple motion periods, near day 219, 238, 244 and 253. The individual flood events and corresponding motion periods are replotted in Figure 3. In addition, two flood events produced a single motion period, near day 241 and 248. The data on the flood events, including total number of monitored periods, and the motion events, including flow at the beginning and end of the event and the number of motion periods, are listed in Table 2. The flood event interval was arbitrarily chosen near the minimum flows between flood events, while the motion event interval is taken as the time between the first and last motion period for a given flood event.

The motion periods shown in Figures 2 and 3 are the recorded data near pulse rate 800 msec in Figure 1. The intermittent timing of the motion events (Fig. 3) result from either rest periods or missing data, therefore some discussion of the reliability of the data is required before progressing to further analysis or interpretation. For this rock, from the start of data acquisition on day 212 to its reach exit date on day 253, a total of 13,144 periods were monitored, each of length 4.5 min. Of this total, due to the extraneous radio interference, 238 periods (1.8%) were either missing or the recorded pulse rate was not near either 1800 msec (rest) or 800 msec (motion) (Fig. 1). The majority of the missing data, 177 periods, occurred during small stream discharges, such as day 212-219 and 220-232, when coarse sediment movement was unlikely. The remaining 61 missing

Table 1 Sediment size parameters and study reach exit dates of the monitored and unmonitored radio-tagged rocks from Lignite Creek 1990.

Radio Frequency	Median Axis (mm)	Final Weight (g)	Final Specific Gravity	Study Reach Exit Date (day)
163.501	60	404.10	2.55	241.460
163.521	71	500.10	2.74	238.817
163.491	75	670.60	2.42	241.458
163.531	73	974.50	3.01	250.548
163.421	64	751.10	2.95	253.751
163.471	74	949.20	2.88	244.467
163.460	75	605.30	2.57	238.754
164.301	74	902.90	2.90	253.729
163.541	73	904.10	2.58	238.510
163.552	85	785.00	2.53	238.489
163.241	80	802.10	2.35	236.089
164.597	98	1027.90	2.58	242.739
163.451	69	815.40	2.46	247.392
163.441	83	639.20	2.54	238.783
163.401	86	763.50	2.56	238.526
164.615	84	1155.00	2.58	238.298
163.412	94	1793.90	2.99	**
164.627	93	1159.30	2.43	235-239*
164.585	87	1337.10	2.84	239-250*
164.572	99	1557.50	2.96	250-264*
164.511	90	1790.80	2.45	235-239*
163.321	--	162.80	2.67	235-239*
163.341	--	144.40	2.21	220-235*
163.281	--	399.20	2.39	235-239*

**Did not exit study reach
*Unmonitored radio-tagged rocks

periods occurred during flows which may have produced sediment movement. The recorded pulse rates from these periods were evaluated individually and all but two periods were categorized as either rest or motion periods. The most common problem that produced missing data in the original filter was a recorded pulse rate that was double either the rest or motion pulse rate. These periods could be reliably reassigned by halfing the recorded pulse rate. Therefore the motion data for this rock appears to be complete and the patterns of the motion periods shown in Figure 3 are reliable and appear to be the actual sequence of rest and motion periods for this particular rock.

The sequence of motion events during a single flood event includes both single 4.5-min periods of motion and multiple consecutive periods of motion. A single motion period indicates that the rock rolled over and stopped and did not move again for two scan periods, or 9 min. Consecutive motion periods indicate that the rock moved once to initially trip the motion sensor, and then moved again sometime before the end on that period (4.5 min) to register a motion pulse rate on the subsequent scan in the scan sequence. The record of motion would be the same whether the rock moved only once at the beginning and once

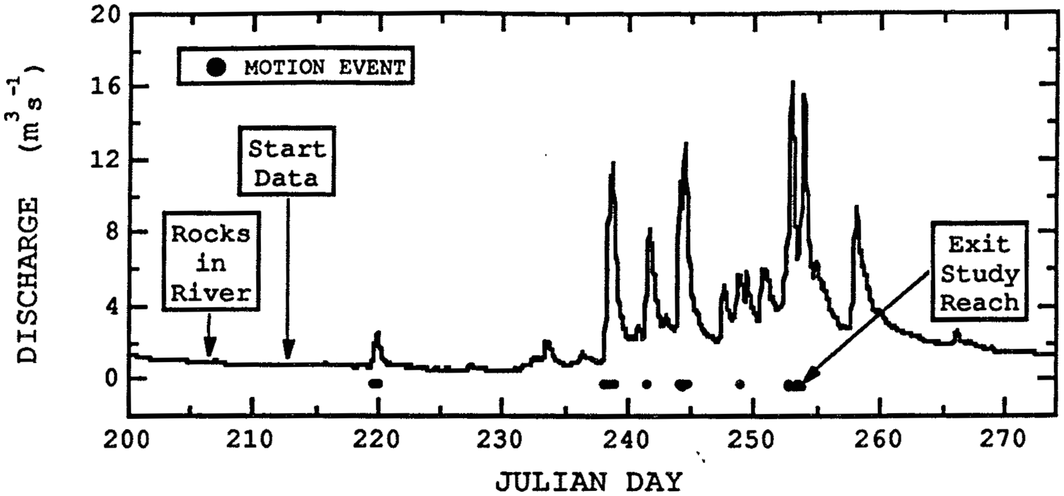


Figure 2. Discharge hydrograph and recorded periods of motion of a single radio-tagged rock (164.301) on Lignite Creek, 1990. Each mark represents the occurrence of motion during a 4.5-min scan interval.

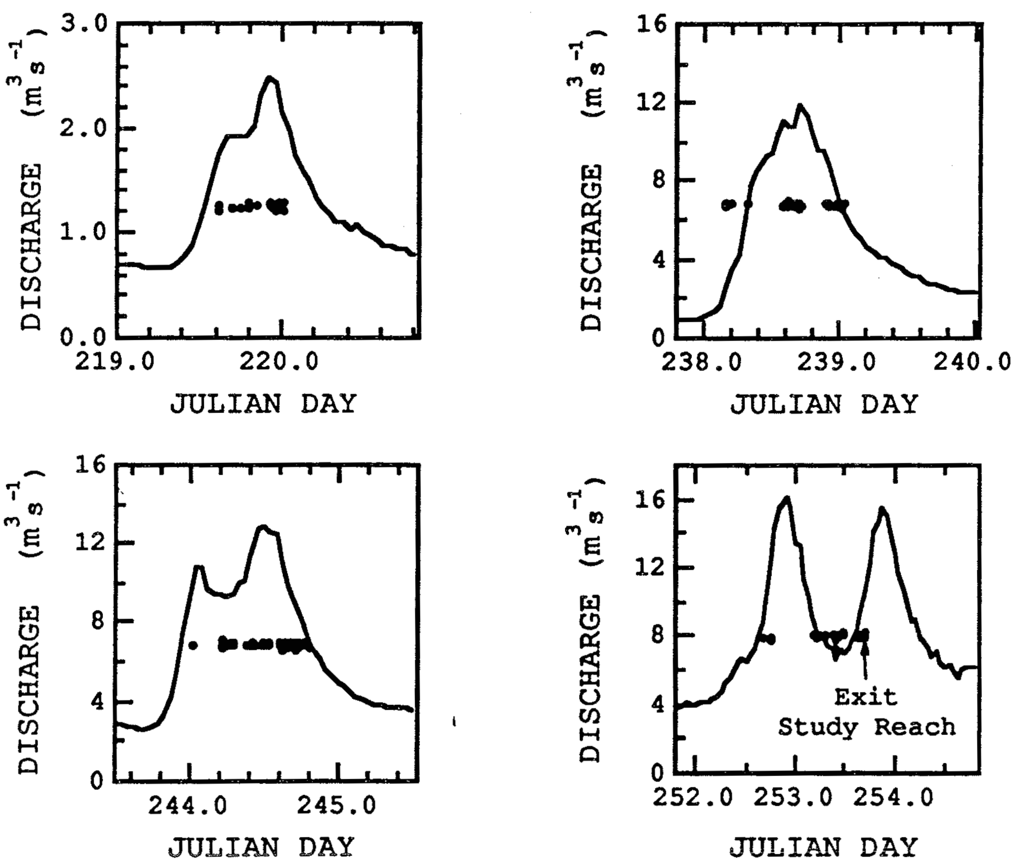


Figure 3. The relationship between motion periods and the four individual flood hydrographs which produced movement of rock 164.301. Circles represent recorded motion during a 4.5-min period. Note vertical and horizontal scales vary on the plots.

Table 2. Flood events and corresponding motion events on Lignite Creek 1990 for rock 164.301.

FLOOD EVENTS		MOTION EVENTS		
Interval (day)	Monitored Periods	Interval (day)	Flow ($m^3\ s^{-1}$)	Motion Periods
219.401		219.614	1.73	
220.401	320	220.014	2.14	22
237.801		238.154	2.37	
239.701	609	239.035	6.41	44
241.251				
242.708	467	241.583	7.54	1
243.701		244.029	10.37	
245.401	545	244.808	7.23	84
247.001				
251.801	1535	248.748	5.77	1
252.010		252.660	8.80	
253.714*	546	253.714*	10.87*	49

*Exited study reach

near the end of the period, or moved continuously throughout the period.

For the first flood event, on day 219 (Fig. 2 and 3), 22 motion periods were recorded, seven single period events and three multiple period events, one of which spanned seven motion periods. A location survey after this flood event measured a travel distance of 2 m for this rock. This results in an average travel distance of 0.091 m per motion period. Based on the median diameter of 74 mm and the total number of motion periods, the movement of this particular rock during this flood event can be characterized as rolling an average of 0.4 revolutions per motion period. Based on the ten motion events, the rock moved an average travel distance of 0.2 m per motion event or 0.86 revolutions per event. These are averages and do not account for sliding or bouncing, nor do they take into account the possibility that the majority of travel may have taken place during one event, perhaps the event of seven consecutive motion periods.

The frequency distribution of consecutive motion periods for all the motion events of this rock during its transport through the study reach is shown in Figure 4. A total of 201 periods of motion were recorded which were divided into 76 separate motion events, 34 single period events and 42 multiple period events. As was done for the first flood event, a similar analysis of the average travel distance can be performed for the remaining motion events. Because the remaining motion events occurred after day 238, during much higher flood events, all these data are lumped together. A travel distance of 360 m was covered in 179 motion periods, or 66 motion events between day 238 and 253. This results in an average distance of 2.01 m per motion period or 5.45 m per motion event. In terms of revolutions, this is 8.6 and 23.4 revolutions per motion period and motion event respectively. It appears that the higher flows following

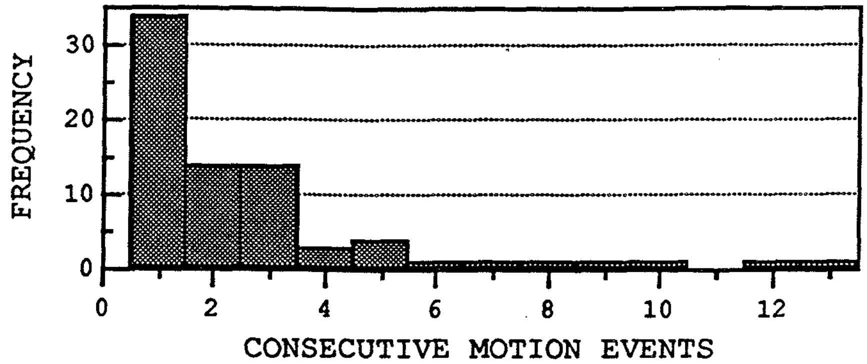


Figure 4. Frequency distribution of consecutive motion events (4.5-min period) for rock 164.301 transported through the study reach of 362 m.

day 238 resulted in longer travel distance (i.e., larger step lengths) per motion period of the rock. Further analysis along these lines await additional data from rocks of various sizes and flow conditions.

An additional observation of interest is the relation between flows at the initiation of motion and the end of motion for the flood events. With the exception of the single period motion event of day 248, motion did not begin during a flood event until flow equaled or exceeded the flow at which motion ended during the previous flood event (Table 2). This is offered only as an observation at this time and interpretation awaits the availability of additional data.

TRAVEL TIME OVER A KNOWN TRAVEL DISTANCE

The reach exit dates provide a second type of data from which analysis of travel times of all the radio-tagged rocks can be undertaken. The actual exit times from the study reach were recorded for the monitored rocks and are plotted on the hydrograph in Figure 5. The exit time for the unmonitored rocks was estimated based on the periodic location surveys. The rocks were placed in the river on day 205 and first surveyed on day 212 at which time no transport had taken place. The next survey on day 220, following a small flood event, measured some transport, but none of the radio-tagged rocks had exited the study reach. However, by the next survey on day 235, also following a small flood event, one unmonitored rock had left the study reach. It seems likely that the time of exit occurred during the small flood event on days 232-234. The exit time is estimated to be at the time of peak flow, day 233.5. The exit times for the remaining unmonitored rocks occurred during the large flood events after day 238 and were estimated simply as mid-way between the surveys dates (Table 1).

Two of the radio-tagged rocks had exited the study reach during relatively low flow events prior to day 238. The series of large floods beginning on day 238 was sufficient to transport all but one of the remaining radio-tagged rocks through the study reach. The one rock which was not transported out of the study reach was recovered at the end of the field season in the lee of a large, embedded boulder. In all but two cases, the monitored rocks exited the study reach on the rising limb or near the peak of the individual flood hydrographs.

The exit date is an indication of the time required for an individual sediment particle to travel the distance of the study reach, and, as such, can be used to address the effect of various

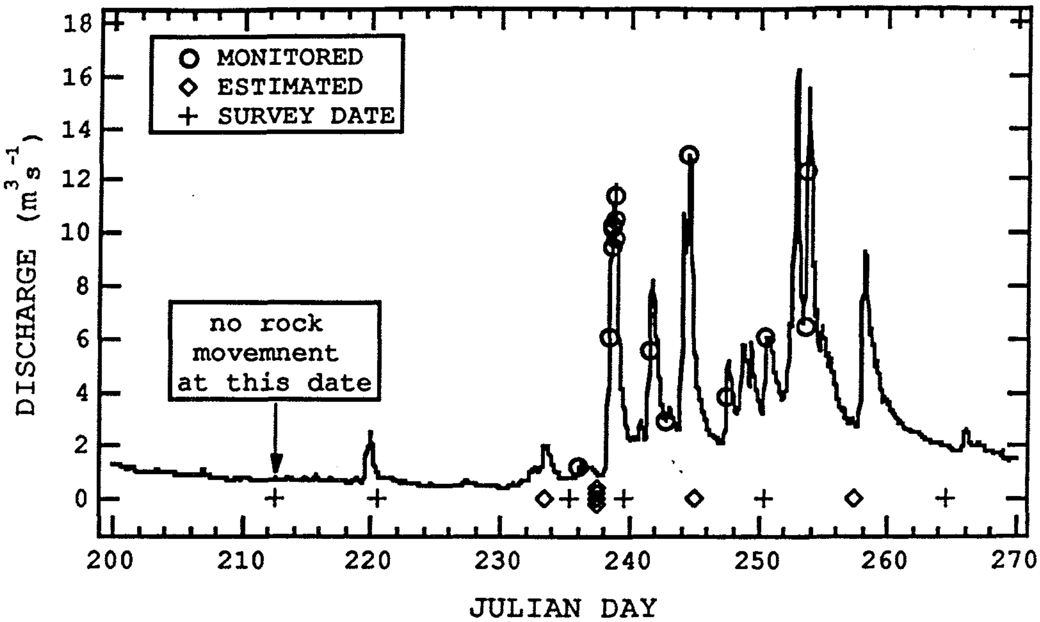


Figure 5. Discharge hydrograph showing the time the radio-tagged rocks exited the study reach (travel distance of 362 m). Monitored rocks are shown on the hydrograph, unmonitored rocks are shown as estimates between the dates of the location surveys.

sediment parameters on the transport time. In Figure 6, the exit date is plotted as a function of the weight of the sediment particles (Table 1). There is some scatter in the data and no clear relationship between weight and travel time is apparent; however, some trends are discernible. All rocks weighing less than 750 g exited the study reach by day 242, the second large flood event. The lightest rock (144 g) was the first to leave the study reach, and the other rock lighter than 200 g may also have exited the study reach before the first large flood event. However, the only other rock known to have exited the study reach before the first large flood event weighed 802.1 g, almost midway through the weight range. The heaviest rock (1793.9 g) did not exit the study reach (traveled about half the length), although a rock of nearly the same weight (1790.8 g) exited the study reach during the first large flood event. Rocks of weights between 750 and 1800 g traveled through the study reach with an apparent equal likelihood of exiting the study reach by a given date.

In addition to particle weight, a number of other sediment parameters were investigated to find a relation between travel time and the material being transported. Various size and shape parameters (i.e., length, flatness ratio, volume ratio) were investigated. All showed considerable scatter except for specific gravity (Fig. 7, Table 1).

The two rocks known to have exited the study reach before day 238, the start of the very large floods, had the lowest specific gravities (Fig. 7). It is also possible that the remaining rock with a specific gravity below 2.4 may have exited the study reach before day 238. Rocks with a specific gravity above 2.9 required the longest time to exit the study reach. At specific gravities between 2.4 and 2.9 there was an apparent equal likelihood that a rock would travel the length of the study reach by a given date at the very large flows experienced in 1990.

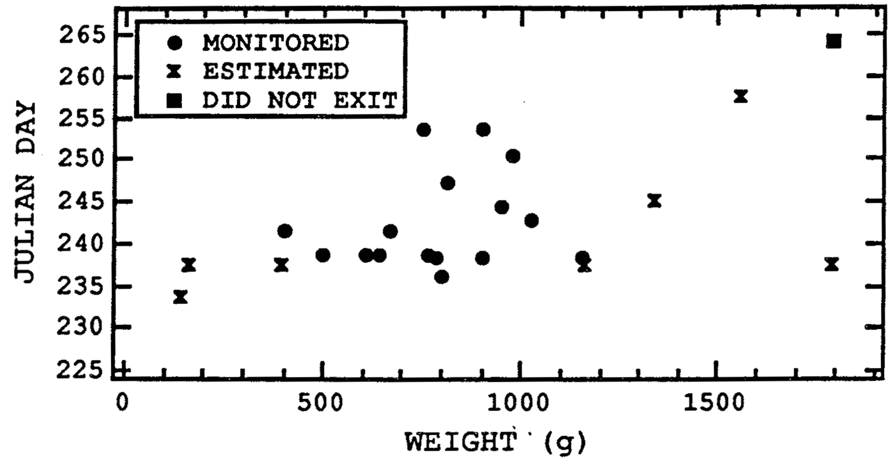


Figure 6. Date at which radio-tagged rocks exited the study reach (travel distance of 362 m) versus weight of individual particles.

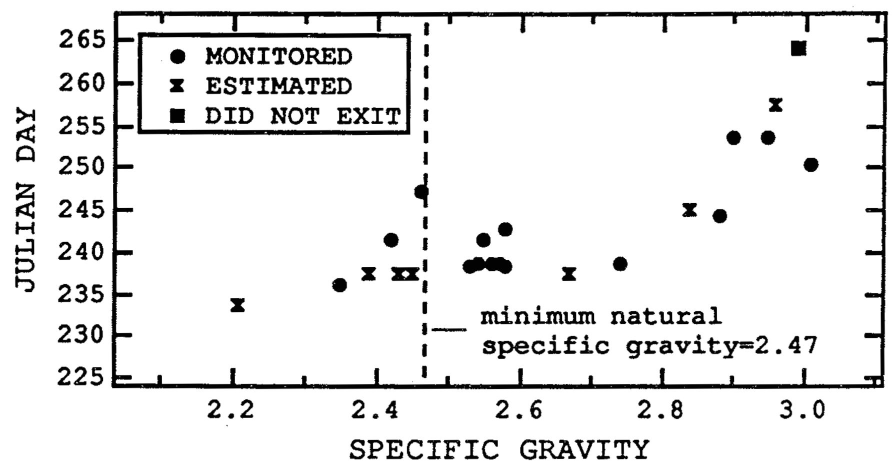


Figure 7. Date at which radio-tagged rocks exited the study reach (travel distance of 362 m) versus the specific gravity of individual particles.

The installation of the radio transmitter in the sediment particles reduced the specific gravity 0% to 18%, depending primarily on the size of the particle; smaller particles were affected more, larger particles were affected less. The minimum natural specific gravity of the radio-tagged rocks was 2.47. No attempt was made to correct to natural specific gravity during this study. Based on this limited data set, it appears that weights of the radio-tagged rocks should be corrected to maintain a minimum specific gravity of 2.4 for this particular stream. As a general rule of thumb in future studies, weights will be corrected to maintain the minimum natural specific gravity of the material in the study stream.

SUMMARY

The use of implantable radio transmitters and an automatic data acquisition system can provide sediment transport data that was not previously available. An initial application of the system has demonstrated that a record of the rest and motion of coarse sediment can be acquired and related to sediment and hydraulic parameters. Travel distances and the timing of transport can also be acquired. Application of the techniques described in this paper will result in a unique data set, that should be beneficial in understanding and modeling coarse sediment transport processes.

REFERENCES

Chacho,E.F.,Jr., Burrows,R.L. & Emmett,W.W. (1989) Detection of coarse sediment movement using radio transmitters. Proceedings of the XXIII Congress of the International Association for Hydraulic Research, The National Research Council Canada, B367-B373.

Emmett,W.W, Burrows,R.L. & Chacho,E.F.,Jr. (In Press) Coarse-particle transport in a gravel-bed river. J of Geomorphology.

Ergenzinger,P.& Schmidt,K.-H. (1990) Stochastic elements of bed load transport in a step-pool mountain river. Hydrology in Mountainous Regions II-Artificial Reservoirs; Water and Slopes IAHS Publ. no. 194, 39-46.

Ergenzinger,P., Schmidt,K.-H. & Busskamp,R. (1989) The Pebble Transmitter System (PETS): first results of a technique for studying coarse material erosion, transport and deposition.Zeitschrift fur Geomorphologie N.F. 33: 4:503-508.

PROBLEMS WITH NUMERICAL MODELING OF GRAVEL-BED RIVERS

Ronald R. Copeland and William A. Thomas
Waterways Experiment Station

Setting up the Geometry

Inherent to one-dimensional numerical models are the assumptions that hydraulic parameters can be assigned a characteristic average value, and that the flow is gradually varied so that the equations of motion can be applied between designated cross-sections. These assumptions are more severely strained in steep gravel-bed streams than in larger, more uniform, sand-bed streams. As a result, more careful attention to numerical model input must be exercised, and more judgement must be applied when interpreting calculated results.

Gravel-bed rivers generally are shallow, subject to braiding, and may have independent flow in various parts of the cross-section. Using average velocity, in streams with high and low velocity cells, sediment transport will be underestimated, especially at lower flows. Figure 1 shows typical variability in depth and differences from average depth.

Gravel-bed rivers are characterized by wide variations in channel width; irregular banks; and frequent variation in depths, including riffle and pool sequences and long bed-forms or debris trains. These characteristics may produce wide variations in local energy slopes that are significantly less than the average slope obtained from topo maps. Most sediment transport functions are very sensitive to slope. Grant, Swanson, and Wolman, (1990) reported on the variable longitudinal slopes in high-gradient streams in the Oregon Cascades. Slopes in pools were about 0.5 percent, in riffles between 0.5 and 1.5 percent, increasing to between about 1.5 and 3.5 percent in rapids, and between 3 and 7 percent in cascades, and up to 40 percent in steps. The average slopes of the streams were between 2 and 4 percent. Emmett, Myrick, and Meade (1982) collected an extensive data set on the East Fork River in Wyoming, and reported significant variation in local slopes over riffles and through pools (figure 2).

Cross-section locations for a numerical model must be selected carefully. A comprehensive model would require cross-sections wherever channel shape changed. This would include every riffle and pool, every expansion and contraction, and at every significant change in longitudinal slope. Intermediate cross-sections would be necessary if changes in energy gradient were large enough to violate the assumption of gradually varied flow. Special care must be taken to insure that cross-sections with significant two-dimensional flow patterns are either modified or not included in the model. This would include sections where eddies develop or that include "dead-water" areas. Such a comprehensive model could require many cross-sections with very short distances between sections. Short distances must be coupled with short time steps in the numerical model. The ability to construct such a comprehensive numerical model would require that the hydraulic engineer be involved in the initial laying out of survey sections.

An alternative to the comprehensive numerical model is a model of selected cross-sections. Ideally these should be located at "average" channel sections, such as a crossing or riffle. However, the hydraulic engineer frequently must use available cross-sections, the locations of which were selected by others, based typically on a pre-established distance and accessibility. In this case, the assigned roughness coefficients in the numerical model must account for the irregularities in the channel between selected cross-sections. Roughness coefficients in the model may vary significantly between cross-sections and with discharge. Adjustment of roughness coefficients must be based on measured data from a know high-water event.

Assigning Input Variables

Boundary roughness is difficult to assign in gravel-bed rivers due to the many factors that influence the Manning's roughness coefficient. These include expanding and contracting flow, both vertically and laterally; bank roughness; grain roughness; bed form roughness; obstructions such as large boulders or fallen trees; and sinuosity. Estimating roughness from grain roughness alone is risky. Bray (1982), using 67 gravel bed streams in Alberta, indicated that predicting n on basis of bed particle size were about 50% too low. Roughness coefficients for a gravel bed in a concrete channel in Corte Madera, California had to be increased 60 percent to account for form losses and sinuosity (Copeland and Thomas 1989).

Onsite surveys and 75 measurements of discharge were made on 21 high-gradient streams (slopes greater than 0.002 ft/ft) as reported by Jarrett (1984). The results were reduced to the following equation:

$$n=0.39\,S^{0.38}R^{-0.16} \tag{1}$$

where S = friction slope
R = hydraulic radius

Notice the absence of particle diameter, or any other physical measure of roughness, in this equation. The resulting n-values are substantially larger than one would get with Limerinos's equation or Keulegan's equations.

Definition of an average bed material gradation is also difficult in a gravel-bed stream. Bed gradation may vary significantly in the lateral, longitudinal, and vertical directions. The bed of a gravel river may have several distinct layers (figure 3). A subsurface layer of relatively well mixed material is frequently overlain by a thin, but coarser cover layer. This layer typically has a thickness equal to the d_{90} of the subsurface layer. Frequently, a discontinuous throughput layer of finer gravel is found on top of the cover layer. Material in the throughput layer is similar to wash load, in that its primary source is upstream supply and/or bank erosion. But a significant portion of throughput is composed of material found in the bed and it may move as bedload. At low flow, throughput may be the only sediment material moving in the stream. Under these conditions, the availability of throughput for transport is limited by the coarse cover layer beneath and the percent of the bed covered by throughput.

Numerical models, that are primarily concerned with simulation of flood events, should use the gradation of the sublayer material as input for the model. It is important to include the maximum size that occurs in the cover layer, in order to allow the model's armoring algorithm to properly simulate hydraulic sorting. This can be accomplished by taking a relatively deep ($20d_{armor}$) sample which includes the armor layer itself. The sample size should be large enough so that the maximum diameter particle comprises less than 1 percent of the total sample.

In some cases, lateral variation in bed material gradation may be so significant that average hydraulic parameters may not be appropriate. Hudson (1983) reported data in the Elbow River in Alberta that demonstrated significant sediment transport on a bar, but no movement in the deeper portion of the channel (figure 4). This was attributed to a much finer bed material present on the bar.

Sediment inflow data is frequently unavailable at the upstream end of the study reach. Even when data is available, it is usually inadequate to define the inflow for each size class and for the total range of discharges. Bedload moves in waves, even with steady flow (figure 5), so that a large number of samples is required to establish average inflow. Generally, the best that we can do is to assume equilibrium transport at the upstream boundary and calculate sediment inflow to the numerical model. If historical bed change data is available, it can be used to adjust sediment inflow.

Sediment Transport Function

Probably the most difficult problem in modelling gravel-bed streams is the lack of a generally applicable sediment transport function. All of currently used sediment transport functions are based on equilibrium transport conditions, which assumes all gravel particles are equally mobile, and average hydraulic parameters. These equations do not apply to calculations for throughput or when a cover layer exists. Data used in their development came from regular cross-sections and/or flumes.

Most gravel-bed functions include the notion of a critical shear stress that defines the beginning of motion. Shields (ASCE 1975) did the classical work establishing a relationship between grain Reynold's number and a dimensionaless critical shear parameter.

$$\frac{\tau_c}{(\gamma_s-\gamma)\,d_{eff}} = f(R_*) \qquad R_* = \frac{U_* d_{eff}}{\nu} \tag{2}$$

Where: τ_c is critical shear stress, γ_s is specific weight of sediment, γ is specific weight of water, U_* is shear velocity, and ν is kinematic viscosity. The function has a constant value when R_* is greater than 300. Sheilds used uniform bed material, measured sediment transport at decreasing levels of bed shear stress, and determined the point of critical shear stress by extrapolation to zero transport. One problem with this approach was that bed forms developed with sediment transport. A portion of the total shear stress was therefore required to overcome the form roughness and the calculated critical shear stress for grain movement was too high. Gessler (1971) reanalyzed Shield's data so that the critical Shields parameter represented only the grain shear stress (figure 6).

Some investigators propose that the critical shear stress for cessation of motion is significantly less than for initiation of motion. This is not accounted for in sediment transport equations and may result in underestimating transport at lower intensity flows. (Note: This notion seems to violate Einstein's (1950) observation that in the equilibrium case there are as many sediment particles depositing as there are being eroded and transported away.)

Critical shear stress is defined as a function of the critical Shield's parameter, θ_c.

$$\Theta_c = \frac{\tau_c}{(\gamma_s - \gamma)\, d_i} \tag{3}$$

Meyer-Peter Muller (1948) and Gessler (1971) determined that the critical Sheilds' parameter for a sediment mixture of gravels was 0.047. Neill (1968) determined, from his data, that in gravel mixtures, most of the particles become mobile when θ_{50} = 0.030. Andrews (1983) found a slight difference in critical shear stresses in a mixture and presented the following equations:

$$\Theta_{ci} = 0.0834 \left(\frac{d_i}{d_{50}}\right)^{-0.872} \tag{4}$$

$$\tau_{ci} = 0.0834\,(\gamma_s-\gamma)\,d_{50}^{0.872}\,d_i^{0.128}$$

Where the i subscript indicates the parameters value for size class i, and d_{50} is the median diameter of the subsurface material. The minimum value for θ_{ci} was found to be 0.020. According to Andrews, the critical shear stress for individual particles has a very small range, and therefore the entire bed becomes mobilized at the same shear stress.

The Shield's parameter not only varies with size class, but also varies with the intensity of the flow. Paintal (1971) measured bed load transport in his flume at shear stresses well below generally accepted critical values, as shown in figure 7.

Experimental data demonstrates the problems that may be encountered when using a sediment transport equation with the critical shear stress as a parameter. This can especially be significant when the shear stress flow is only slightly greater or less than "critical", or when the bed material gradation is well graded or bimodal.

A commonly used critical shear stress equation for gravel-bed streams is the Meyer-Peter Muller (1948) function. This function was developed from flume data with graded material ranging in size between 0.4 mm and 28 mm.

$$\left(\frac{g_{sb}(\gamma_s-\gamma)}{\gamma_s}\right)^{\frac{2}{3}}\left(\frac{\gamma}{g}\right)^{\frac{1}{3}} 0.25 = \left(\frac{k}{k'}\right)^{\frac{3}{2}}\gamma RS - 0.047\,(\gamma_s-\gamma)\,d_m \tag{5}$$

where: g_{sb} is the bedload discharge per unit width of channel, g is the acceleration of gravity, k is the reciprocal of Manning's coefficient, k'is the reciprocal of Manning's coefficient attributed to grain roughness, R is the hydraulic radius, S is the average energy slope, $d_m = \Sigma p_i d_i$, p_i is the percentage of size class i in the bed, and d_i is the mean size of size class i.

Regression equations for sediment transport are limited by the data base from which they were developed. One such equation was developed by Yang (1984).

$$\log C = 6.681 - 0.633 \log \frac{\omega d}{\nu} - 4.816 \log \frac{U_*}{\omega} + \left(2.784 - 0.305 \log \frac{\omega d}{\nu} - 0.282 \log \frac{U_*}{\omega}\right) \log \left(\frac{VS}{\omega} - \frac{V_{cr}S}{\omega}\right) \tag{6}$$

where: C is the sediment concentration, ω is the fall velocity, d is the grain diameter, V is the flow velocity, and V_{cr} is the critical flow velocity defined by equation 7.

$$\frac{V_{cr}}{\omega} = \frac{2.5}{\log\left(\frac{U_* d}{\nu}\right) - 0.06} \; ; \; 1.2 < \frac{U_* d}{\nu} < 70$$

$$\frac{V_{cr}}{\omega} = 2.05 \; ; \; 70 < \frac{U_* d}{\nu} \tag{7}$$

Its data base was limited to uniform gravel sizes between 2.5 and 7 mm. Note that no allowance is made for reduction in the applied shear stress due to bed forms. This function has the unique characteristic of increasing transport with increasing grain size.

Einstein's (1950) bedload function is not dependant on critical shear stress or critical velocity. Instead it is a probabilistic approach.

$$\Phi_{*i} = f(\Psi_{*i})$$

$$\Phi_{*i} = \frac{g_{bi}}{p_i \gamma_s}\left(\frac{\gamma}{(\gamma_s - \gamma)\,(g d_i)^3}\right)^{0.5} \tag{8}$$

$$\Psi_{*i} = \xi_i\, Y\left(\frac{\log 10.6}{\log(10.6\, d_x/\Delta)}\right)^2 \frac{(\gamma_s - \gamma)\, d_i}{\gamma R' S}$$

where: Φ_{*i} is the the bedload intensity parameter, Ψ_{*i} is the flow intensity parameter, g_{bi} is the sediment transport per unit width, p_i is the percentage of size class i in the bed, ξ_i is the hiding factor, Y is the lift coefficient, and Δ is the characteristic grain size for the mixture. The relationship between Φ_* and Ψ_* is determined from empirical data (figure 8). Adjustments to the lift coefficient due to a mixture are accounted for by Y. Hiding of the smaller particles in a mixture is accounted for by the hiding factor ξ_i. This parameter is very sensitive to the characteristic of the mixture and several investigators have revised Einstein's original hiding function in order to match their particular data sets (figure 9 Shen and Lu 1983).

Even after all these years of investigation, there is still no "best" equation for sediment transport. Although there currently is no alternative, there remains a great deal of uncertainty with respect to extrapolating existing equations into the gravel/cobble bed particle sizes and mixtures.

The Effect of Armoring

Many natural gravel-bed rivers possess a coarse surface layer similar to the armor layers that develop downstream from dams. However, the winnowing process associated with immobile armor does not appear to be a reasonable explanation of the process occurring in a natural stream in equilibrium. Milhous (1973) collected data on Oak Creek, a small gravel-bed stream in Oregon. Bed shear stresses on this creek rarely exceeded the critical shear stress by a factor of more than 2 or 3. He found a coarse surface layer with a median grain size about twice that of the subsurface material. Bedload measurements with a vortex-tube extractor indicated movement of all the size classes present in the subsurface layer and a bedload gradation similar to the gradation of the subsurface layer.

The concept of a mobile coarse surface layer, that exists even at high flow, and which serves as a sediment transport regulator by exposing proportionally more coarse grains to the flow, has been called the equal mobility hypothesis (Parker and Klingeman 1982) The concept is applicable to bedload transport streams where the bed shear stresses do not exceed the critical bed shear stresses by more than a factor of 2 or 3. Parker and Klingeman deduced that the grains in the coarse-surface layer of a gravel-bed river move continuously, but sporadically. The coarser material on the surface serves to shield the finer material below the surface so that the probability of entrainment of fine material is decreased. In addition, the greater exposure of the coarsest sizes results in greater potential for their transport. As a result, the gradation of the subsurface material is essentially equal to the gradation of the bedload. The equal mobility concept has been compared to the hiding concept proposed by Einstein (Andrews and Parker 1987).

Jain (1990) tied together the armoring process downstream from dams and the equal mobility ideas using conceptual experiments. He first viewed the relationship of sediment discharge (bedload) and increasing shear stress with time (figure 10). As shear stress increases the bed initially coarsens as immobile particles are left on the surface as armor. Bedload discharge is highest at time zero and decreases until only immobile particles are left on the bed and transport ceases. With increasing shear stress, a point is reached where all particles are mobile, but only sporadically. Once this condition is reached, the bed becomes less coarse because critical shear stress has been exceeded for all particles. Eventually, the shear stress is large enough to completely destroy the coarse surface layer and the surface has the same gradation as the parent bed material.

Kuhnle (1989) conducted flume studies at the Agricultural Research Service Laboratory in Oxford, Mississippi, to determine the applicability of the equal mobility concept at high shear stresses. He demonstrated that a flume with a gravel bed could have the characteristics of a sand-bed stream. His flume was a sediment feed flume. He found samples of the bed surface that developed in his flume were actually finer than the original bed sediment, this means that the bed composition was determined by the sediment feed. Bed forms, about 2-4 mm high and 1-3 m long, were observed in the flume and were composed predominantly of 2-4 mm grains. Larger material (8-32 mm) was transported at high velocities over the bedforms. Test results demonstrated that the mobile-bed armor layer in a laboratory flume gradually disappeared as the effective shear stress was increased, while the size distribution of the transported sediment was the same in all runs. This corroborates the prediction of Parker that at high bedload transport rates the mobility difference of coarse and fine grain sizes in a gravel sediment mixture disappears and a coarse armor layer is no longer required for equilibrium transport of all sizes.

Conclusions

Sediment transport capacity varies laterally across the channel and thus the assumption of average hydraulic parameters in a one-dimensional numerical model may result in an underestimation of sediment transport.

Due to irregularities in channel geometry, a large number of cross-sections are required for an accurate numerical model. An alternative approach is to use fewer cross-sections and adjust the numerical model using roughness coefficients.

Roughness coefficients must account for losses due to channel irregularities, bank roughness, form roughness, obstructions, and sinuosity, in addition to the grain roughness.

For numerical simulation of flood events, the gradation of the subsurface layer is most appropriate for input to the numerical model.

Care must be taken to consider lateral variations in bed material gradation, that may make the use of an "average" gradation inappropriate.

Sediment inflow to the numerical model will usually have to be calculated.

There is no generally applicable sediment transport function for gravel-bed rivers. Part of the problem is the inability to define the critical shear stress based on "average" hydraulic conditions, or in a mixture. Another problem is the definition of the percentage of a size class involved in exchange with the water column. This is complicated by the formation of a coarse surface layer or a discontinuous throughput layer.

In spite of these numerous problems, successful numerical model studies of gravel-bed streams have been conducted. However, it is important to recognize the limitations of the numerical model with respect to the natural alluvial processes, and to verify the numerical model performance for each study.

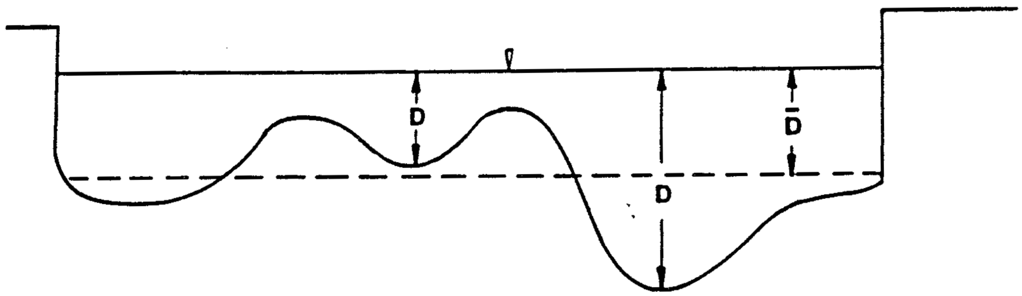


Figure 1. Effects of varying depth (D) in a cross-section on distribution of local shear stress in a wide channel (Carson and Griffiths 1987).

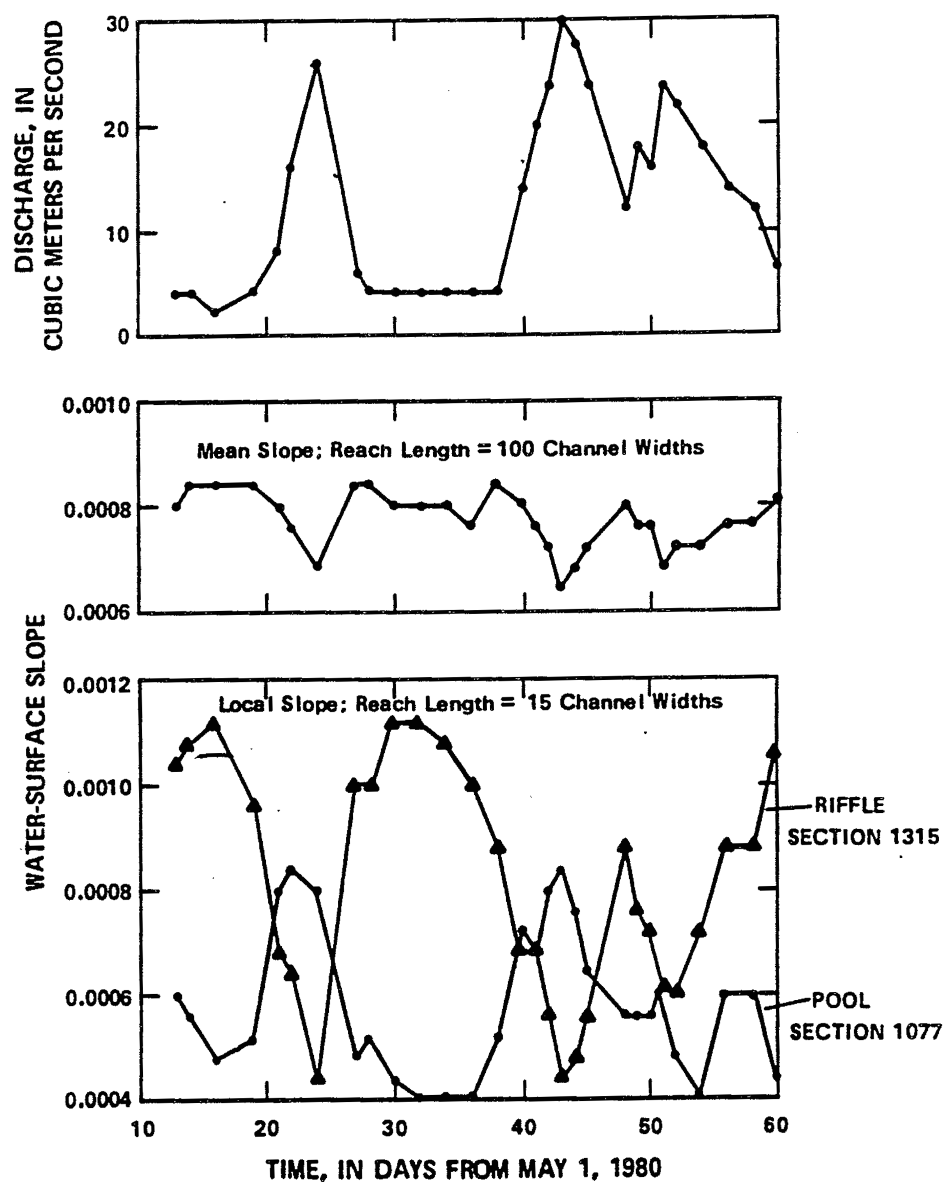


Figure 2. Variation of local water-surface slope with discharge for a pool and a riffle, East Fork River, Wyoming (Emmett, Myrick, and Meade 1982).

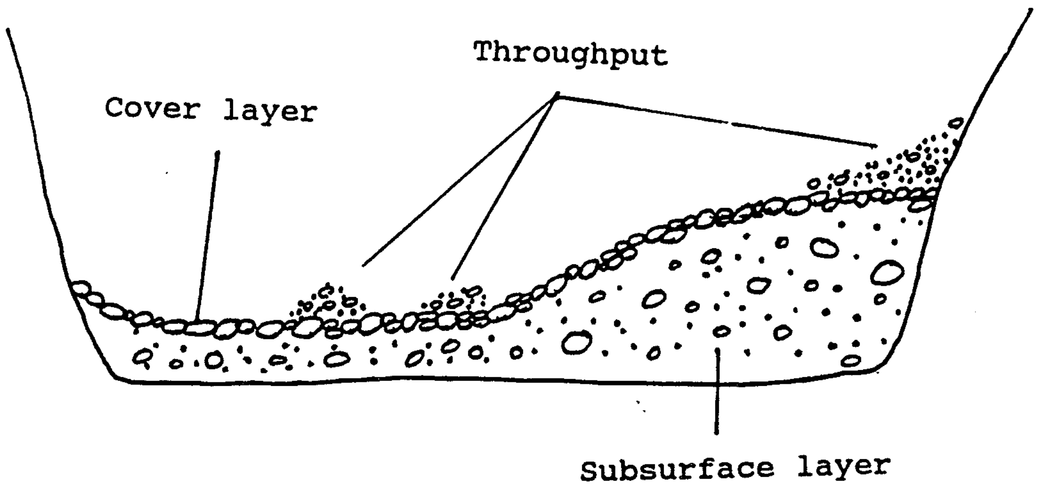


Figure 3. Typically layering of bed material in a gravel-bed stream.

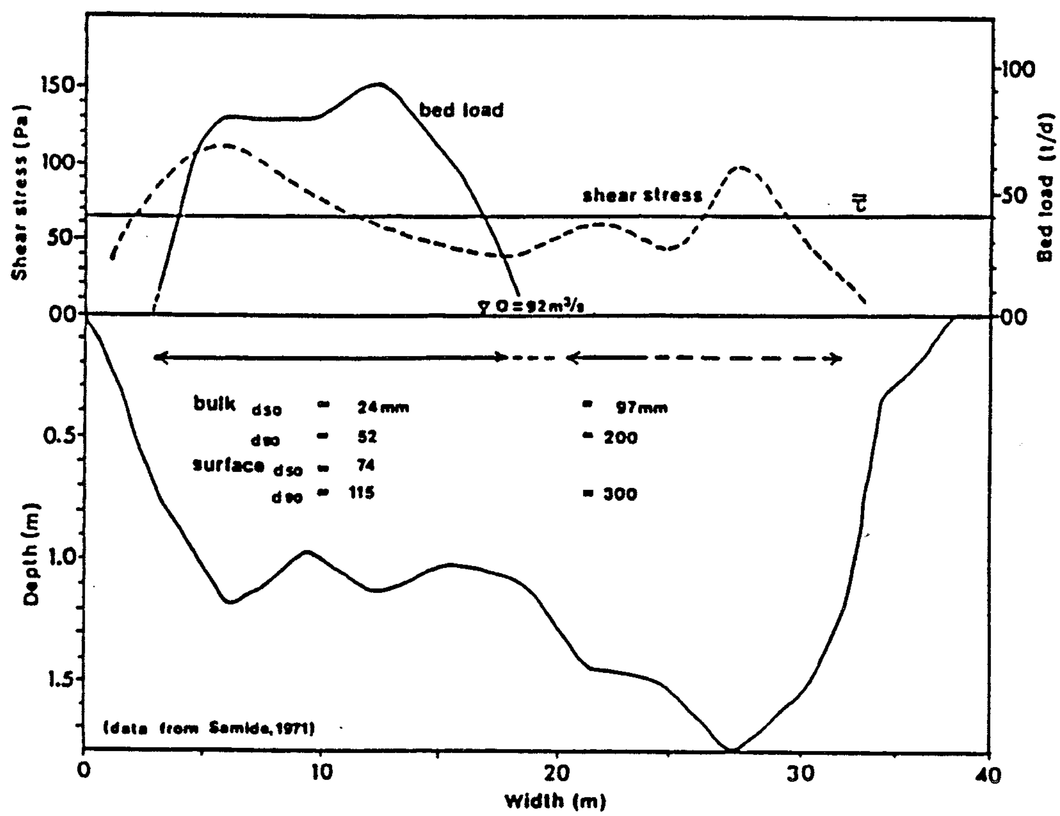


Figure 4. Cross-Sectional variation in bedload transport rate and shear stress, Elbow River, Canada. (Hudson 1983)

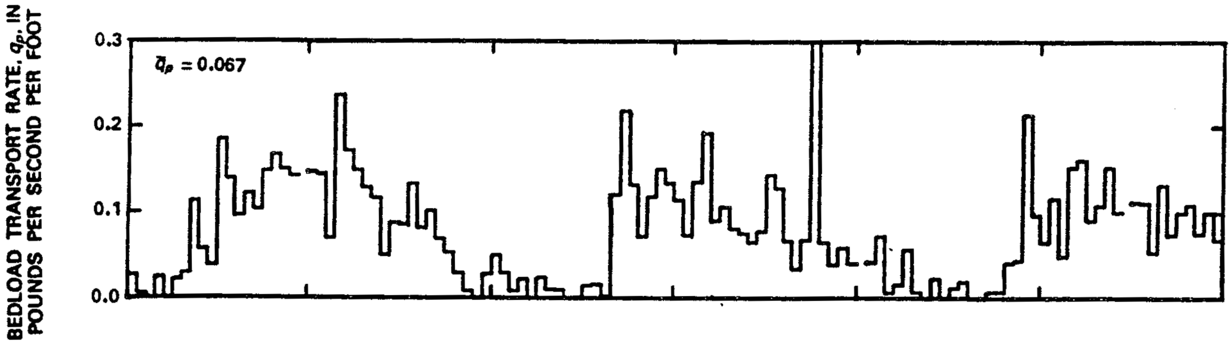


Figure 5. Temporal records of bedload-transport rates with 6.5 mm bed material. (Hubbell et al 1987)

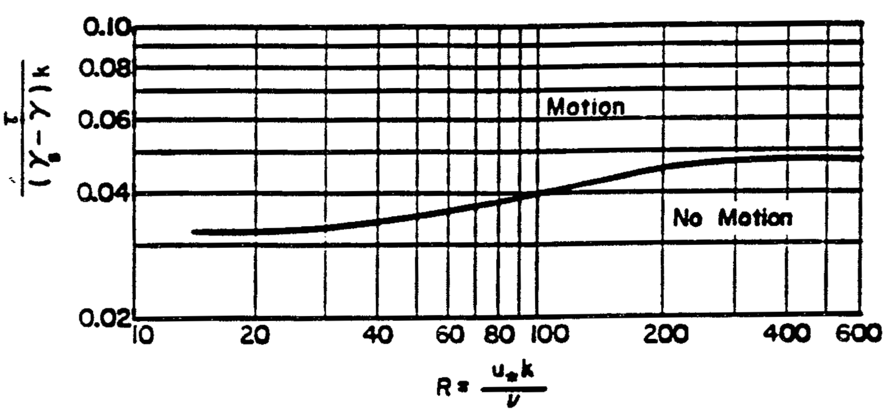


Figure 6. Shield's diagram. (Gessler 1971)

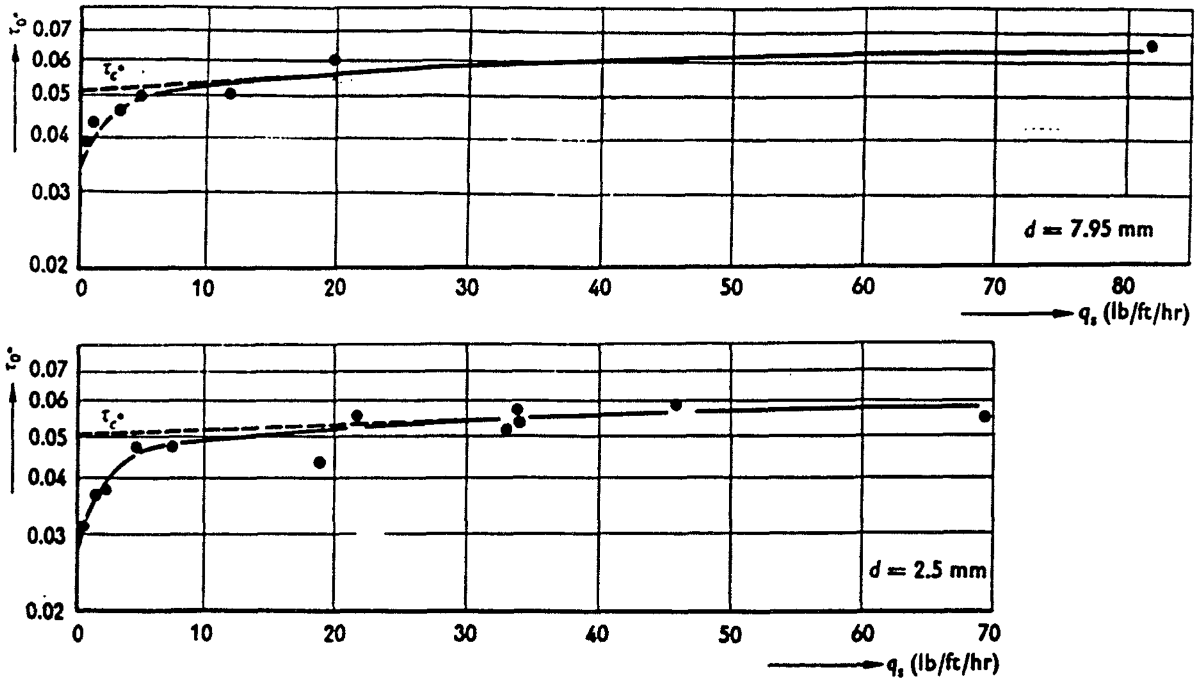


Figure 7. Determination of critical shear stress. (Painetal 1971)

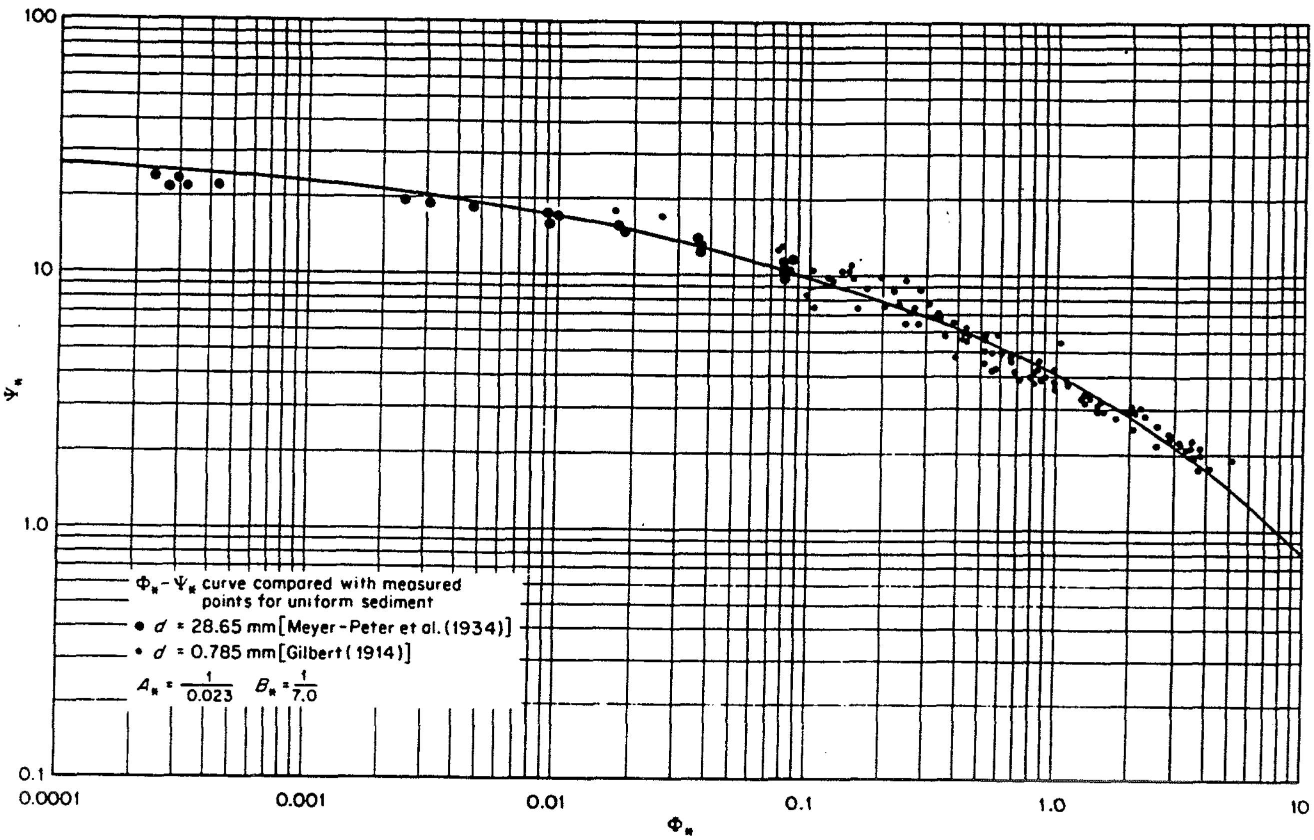


Figure 8. Plot of Einstein's functions; Φ_* vs. Ψ_*. (Einstein 1950)

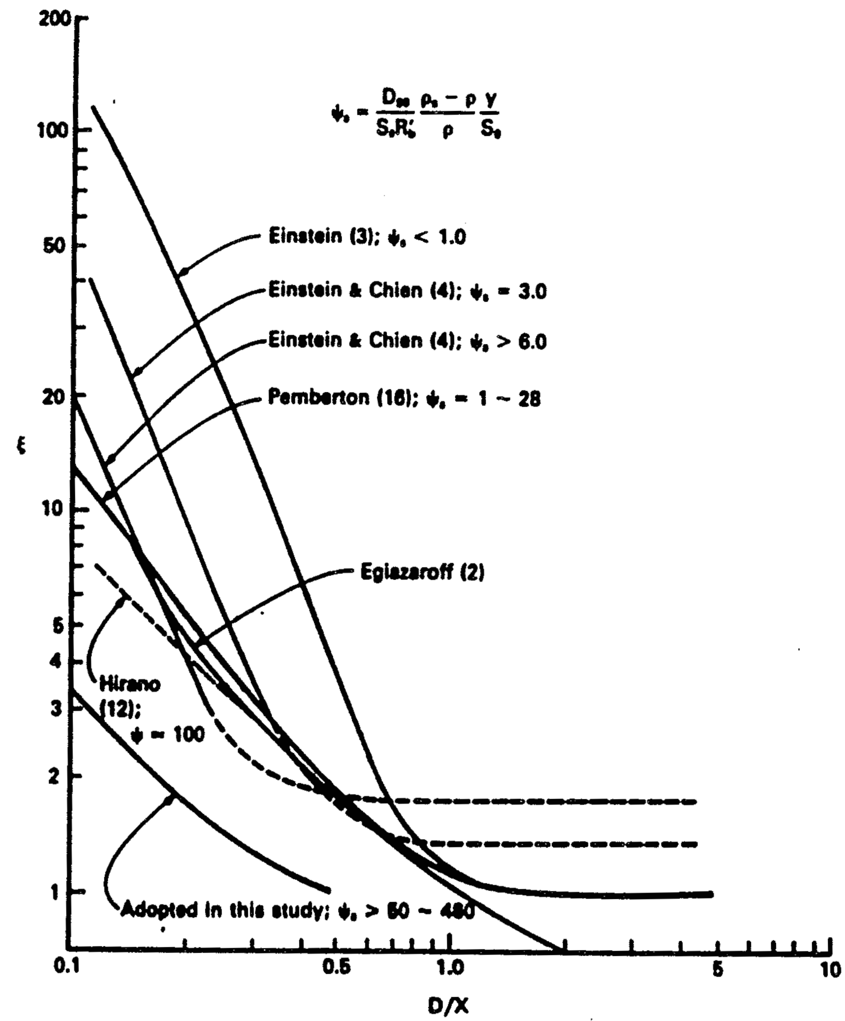
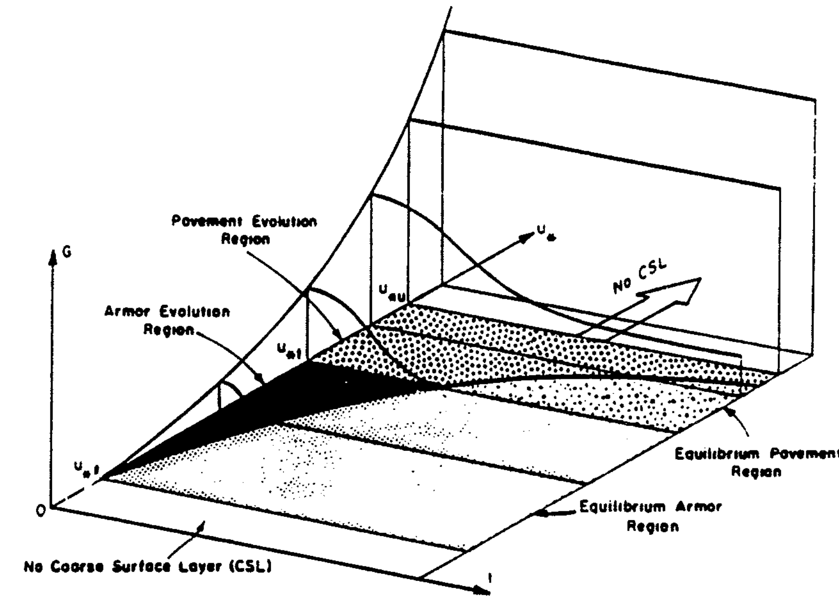
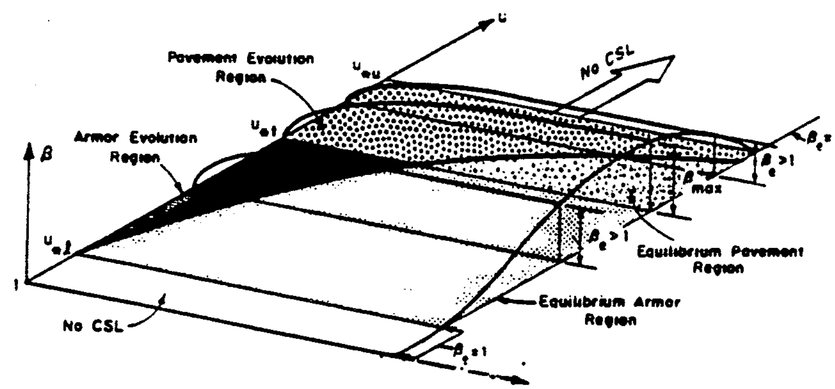


Figure 9. Hiding factor of sediment particles in mixture for predicting bed armoring layer only. (Shen and Lu 1983)



Variation of Eroded-Material Discharge during Surface-Layer Formation



Degree of Coarseness of Surface Layer

Figure 10. Jain's (1990) Concept of Armor Layer Development and Disintegration.

REFERENCES

Andrews, E.D. 1983(Oct). "Entrainment of Gravel from Naturally Sorted Riverbed Material," Geological Society of America Bulletin, Vol 94, pp. 1225-1531.

__________, and Parker, Gary 1987. "Formation of a Coarse Surface Layer as the Response to Gravel Mobility," Sediment Transport in Gravel-Bed Rivers, Chapter 10, Ed. Thorne, C.R.; Bathurst, J.C.; and Hey, R.D., John Wiley & Sons, pp. 269-325.

American Society of Civil Engineers 1975. Sedimentation Engineering, Manual on Engineering Practice No. 54, Vanoni, V.A. editor, New York.

Bray, D.I. 1982. "Flow Resistance in Gravel Bed Rivers," Gravel Bed Rivers, Hey,R.D.; Bathurst, J.C.; and Thorne, C.R. (eds), John Wiley and Sons, New York, pp 109-137.

Carson, Michael A.; and Griffiths, George A. 1987. "Bedload Transport in Gravel Channel," Journal of Hydrology, New Zealand, Vol 26., No. 1.

Copeland R.R., and Thomas, W.A. 1989 (April). "Corte Madera Creek Sedimentation Study; Numerical Model Investigation," Technical Report HL-89-6, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Einstein, Hans Albert 1950 (September). "The Bed-Load Function for Sediment Transportation in Open Channel Flows," Technical Bulletin No. 1026, U.S. Department of Agriculture, Soil Conservation Service, Washington, D.C.

Emmett, W.W.; Myrick, R.M.; Meade, R.H. 1982. Field Data Describing the Movement and Storage of Sediment in the East Fork River, Wyoming, U.S. Geological Survey Open-File Report 82-359, Denver Colorado.

Gessler, Johannes 1971. "Beginning and Ceasing of Sediment Motion," Chapter 7, and "Aggradation and Degradation," Chapter 8, River Mechanics, Vol. I, Hseih W. Shen, editor and publisher, Fort Collins, Colorado.

Grant, G.E.; Swanson, F.J.; and Wolman, M.G. 1990 (March). "Pattern and origin of stepped-bed morphology in high-gradient streams, Western Cascades, Oregon," Geological Society of America Bulletin, Vol 102, pp.340-352.

Hubbell, D.W.; Stevens Jr.,H.H.; Skinner, J.V.; and Beverage, J.P. 1987. Laboratory Data on Coarse-Sediment Transport for Bedload-Sampler Calibrations, U.S. Geological Survey Water-Supply Paper 2299, Denver CO.

Hudson, H.R. 1983. Elbow River Hydrology and Sediment Transport, PhD Thesis, University of Alberta, Edmonton.

Jain, Subhash C. 1990. "Armor or Pavement," Journal of Hydraulic Engineering, ASCE, Vol. 116. No. 3, pp. 436-440.

Jarrett, Robert D. 1984. "Hydraulics of High-Gradient Streams," Journal of Hydraulic Engineering, Vol. 110, No. 11, American Society of Civil Engineers, pp. 1519-1539.

Kuhnle, Roger A. 1989 (Jun). "Bed-Surface Size Changes in Gravel-Bed Channel," Journal of Hydraulic Engineering, ASCE, Vol. 115, No. 6, pp. 731-743.

Meyer-Peter, E., and Muller, R. 1948. "Formulas for Bed-Load Transport," Second Meeting of the International Association for Hydraulics Research, Stockholm, Sweden, Appendix 2, pp. 39-64.

Milhous, R.T. 1973. Sediment Transport in a Gravel Bottomed Stream, Phd. thesis, Oregon State University, Corvallis, Oregon.

Neill, C.R. 1968. A Re-Examination of the Beginning of Movement for Coarse Granular Bed Materials, Hydraulic Research Station, Wallingford, England.

Paintal, A.S. 1971. "Concept of Critical Shear Stress in Loose Boundary Open Channels," Journal of Hydraulic Research, No.1, pp. 90-113.

Parker, Gary, and Klingeman, Peter C. 1982 (Oct.). "On Why Gravel Bed Streams are Paved," Water Resources Research, Vol. 18, No. 5, pp. 1409-1423.

Shen, Hseih W., and Lu, Jau-Yau, 1983 (April). "Development and Prediction of Bed Armoring," Journal of Hydraulic Engineering, American Society of Civil Engineers, Vol. 109, No. 4, pp. 611-629.

Yang, Chih Ted 1984 (December). "Unit Stream Power Equation for Gravel," Journal of the Hydraulics Division, American Society of Civil Engineers, Vol. 110, No. HY12, pp. 1783-1797.

BED LOAD ROUGHNESS IN SUPERCRITICAL FLOW

by

Scott E. Stonestreet[1], Ronald R. Copeland[2], Darla C. McVan[2]

INTRODUCTION

The US Army Engineer District, Los Angeles, has proposed to improve the lower reach of Mission Creek (US Army Engineer District, Los Angeles, 1986), located in Santa Barbara County, California. A rectangular concrete channel has been designed to convey the 100-year peak flow of 7900 cfs at supercritical flow.

The Mission Creek watershed is capable of supplying large-sized material for sediment transport. Since a debris basin is not included as part of the project, there is a possibility that boulders, with diameters up to 305 mm, may enter the channel. This material, moving as bed load along the channel bottom, could increase the hydraulic roughness to the point where the flow regime may change from supercritical to subcritical flow.

Flume and numerical model investigations have been conducted to evaluate the effect of bedload transport on hydraulic roughness, and the characteristics of the bed-load transport itself. The flume study was conducted at the US Army Engineer Waterways Experiment Station (WES). In conjunction with the flume study, a numerical sedimentation model study was conducted by the US Army Engineer District, Los Angeles.

DESCRIPTION OF STUDY AREA

The Mission Creek watershed comprises about 11.5 square miles and is located in a narrow coastal area which extends from the Santa Ynez mountains on the north to the Pacific Ocean on the South. Mission Creek rises about 3750 feet in elevation and flows about 8 miles to empty into the Pacific Ocean. At approximately the 500 foot elevation, the creek is joined by its main tributary, Rattlesnake Creek. In the headwater areas, stream gradients are as steep as 2600 feet per mile and average 1000 feet per mile. In the lower reaches, on the alluvial plain below the foothills, average slopes are about 150 feet per mile. A profile of Mission and Rattlesnake Creeks is shown on Figure 1.

In 1964, the Los Angeles District constructed two small debris basins in the upper reaches of Mission and Rattlesnake Creeks as an emergency measure to reduce flooding. The location of these basins is shown on Figure 1. These basins are relatively small and do not provide significant protection to the proposed project reach. Additionally, the basins are separated from the project reach by about four miles of natural channel.

Two sections of Mission Creek were improved with concrete-lined, supercritical trapezoidal channels by the state transportation agency in 1934 and 1964 (see Figure 1). To date, the existing channels have functioned satisfactorily without significant maintenance requirements.

[1]Hydraulic Engineer, Los Angeles District, US Army Corps of Engineers

[2]Research Hydraulic Engineer, Waterways Experiment Station, US Army Corps of Engineers

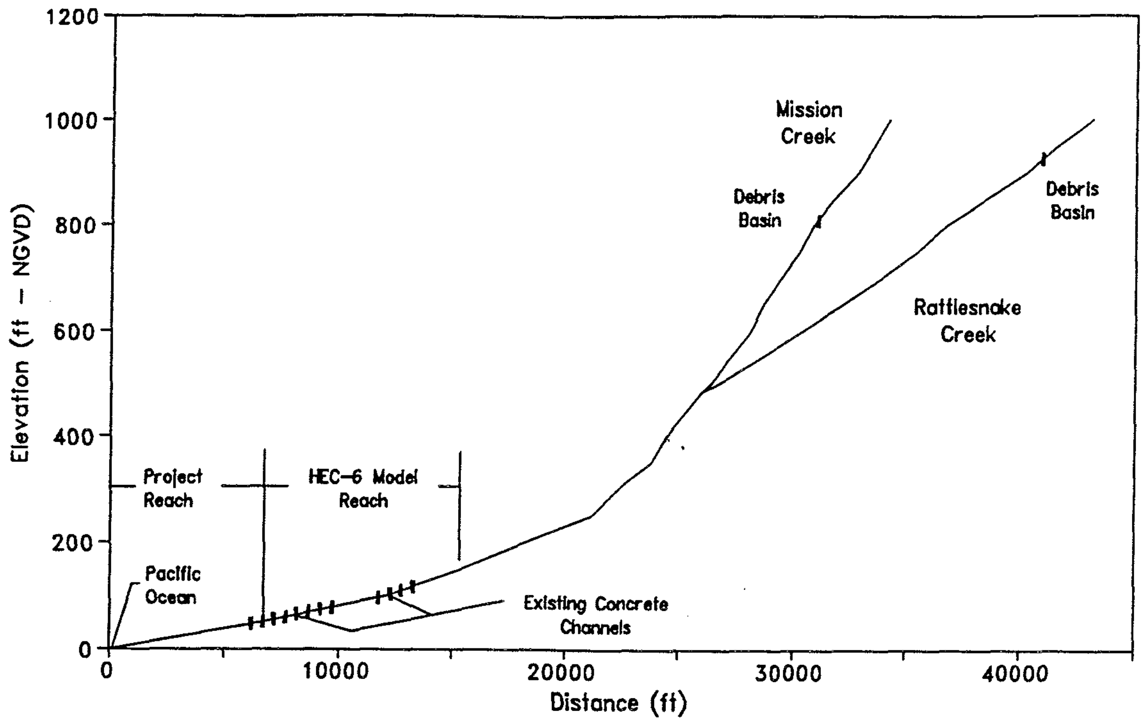


Figure 1. Thalweg profile of Mission and Rattlesnake Creeks

The Corps' proposed concrete channel will adjoin the existing downstream concrete channel and will convey flows 1.1 miles to the Pacific Ocean. The right-of-way along the channel alignment is minimal which necessitated use of the supercritical rectangular channel.

GRADATION ANALYSIS

Sediment samples were collected along the creek and a gradation analysis was performed to determine the grain size distribution and the maximum particle size. Along the channel, a backhoe was utilized to collect the sediment samples since the bed material consisted primarily of boulders and cobbles. At the beach, a crane fitted with a clamshell bucket was used to collect samples due to wet conditions at the site. The sediment gradation for Mission Creek is shown on Figure 2. The maximum grain size of the bed material collected was about 305 mm with an average d_{50} of about 50 mm.

Additional sediment data was provided by a debris deposition study performed for the Los Angeles District (Simons, 1984). Gradations at several locations along the upper reaches of Mission Creek, well upstream of the project reach, were provided. This gradation analysis used the pebble-count method and resulted in a maximum sediment size in the 4 to 8 foot range and a d_{50} of approximately 10 inches (254 mm). As expected, the maximum and average sediment sizes generally decrease in the downstream direction.

INFLOWING LOAD

General Approach. The amount of sediment inflowing to the proposed channel was estimated by routing a 100-year balanced hydrograph through the natural

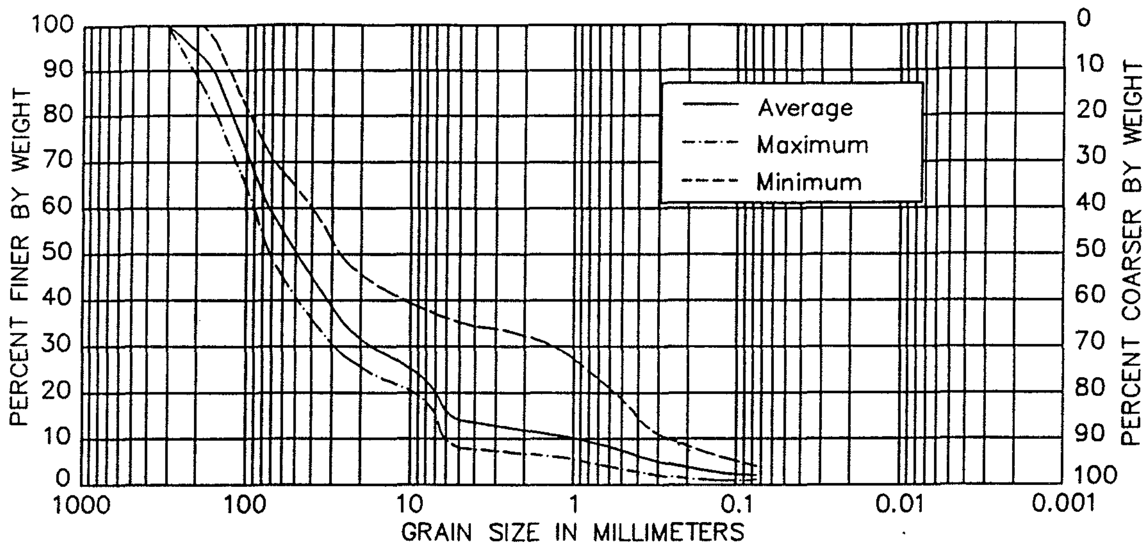


Figure 2. Gradation of bed-material

channel upstream of the project reach. The sediment routing was performed with a special version of TABS-1 (i.e. HEC-6), a one-dimensional numerical model developed at WES, which allows for transport of individual grain sizes larger than 64 mm along with sand and gravel sizes. Equilibrium transport was assumed at the upstream boundary of a 1.8-mile study reach (see Figure 1).

In this analysis, it is expected that the bed-load material will consist primarily of gravel, cobbles, and boulders. Sand and silt sized material should be carried as suspended-load given the high velocities of the relatively steep stream system.

Throughout the sediment supply reach, several constrictive bridges exist in the prototype. In reality, these bridges will obstruct flow and trap significant amounts of sediment and floating debris reducing the amount of sediment which enters the project reach. However, as a conservative measure, the effect of these bridges was not included in the numerical model.

Hydrology. A 60-hour balanced hydrograph was modeled as a histograph with a peak discharge of 6700 cfs. An additional 1200 cfs is generated in the urbanized area downstream of the sediment supply reach and was not included in this model. A constant time step of five minutes was used throughout the simulation.

Sensitivity Analysis. A sensitivity analysis was conducted since prototype data did not exist for model adjustment. The sensitivity analysis consisted of testing various sediment transport functions and then varying the inflowing sediment load for each function. The transport functions included Yang's unit stream power, a combination of the Toffaleti and Schoklitsch functions, a combination of the Toffaleti and Meyer-Peter and Muller functions, and the Meyer-Peter and Muller function by itself. The inflowing sediment load ratio was halved and doubled at the upstream end of the numerical model during the simulations to account for uncertainty in the inflow load.

Results. Results for the sediment routing for the peak discharge are shown in Table 1 and indicate that the concentrations flowing into the concrete channel may vary between 1450 and 17,900 ppm for the total load, in which the bedload (gravel and/or boulder) concentrations vary from 61 to 1030 ppm. These results are for an inflow sediment load ratio of 1.0. Additionally, it appears that the

Table 1. Total and bedload concentrations at peak discharge for different transport functions

Transport Function	Total Load (t/d)	Total Concentration (ppm)	Bedload Load (t/d)	Bedload Concentration (ppm)
Yang	323,700	17,900	1100	61
Toff-Schoklitsch	248,800	13,800	10,500	580
Toff-MPM	160,600	8880	18,600	1030
MPM	26,300	1450	17,200	950

combination of the Toffaleti and Meyer-Peter and Muller transport functions gives the highest or most conservative bedload concentration.

As expected, variation in the sediment load ratio produces slightly different results. For example, using the Toffaleti and Meyer-Peter and Muller combination, an inflow load ratio of 0.5 gives a bedload concentration of about 1000 ppm; whereas, an inflow load ratio of 2 produces a bedload concentration of about 1300 ppm.

PURPOSE OF THE FLUME STUDY

As early as 1946, Vanoni (1946) demonstrated that suspended sand, at concentrations between 1200 and 3300 ppm, caused a reduction of up to 10 percent in effective bed roughness over a flat bed. It has also been demonstrated by many investigators that both sand and gravel bed forms moving along the bottom of a flume or river at a velocity much slower than the flow can significantly increase effective roughness. It is anticipated that flow energy in Mission Creek will be sufficient to prevent the establishment of bed forms at expected rates of bed-load transport. However, it is expected that bed load moving along the bed of the channel at a velocity slightly less than the velocity of the water will introduce some drag and will therefore tend to increase the effective hydraulic roughness. The purpose of the flume study was to quantify this increase and to determine at what concentration bed forms begin to appear.

FLUME STUDIES

The tilting steel flume used in this study is about 80 feet long and 3 feet wide. The flume tests were conducted using steady uniform flow, and were designed to model the prototype's minimum Froude number, slope, and velocity at the flood peak. This was accomplished using a model scale of 1:32.1 based on Froudian criteria. Sediment was introduced into the flume by a constant feed sediment hopper. The motor-operated elevator hopper has a capacity of 22 cu ft and allows for tests of about 45 minutes. Before each test, sediment was placed in the hopper and a vibrating rod was used to consolidate the material. The sediment was leveled with the bottom of the flume bed and then lowered approximately 5 inches below the flume bed. Flow was then introduced into the flume and initial water levels measured without sediment transport. The hopper elevation rate was set to achieve specified concentrations. As the hopper rose, sediment was introduced into the flow and eventually a constant feed rate was achieved.

Water depths in the flume were measured in five stilling wells. The purpose of the stilling wells was to eliminate the difficulty in determining average depth with the waves that are characteristic of rapid flow. The flow exited the flume in free flow.

Roughness Coefficients. The Manning's roughness coefficient was determined by the slope-area method, using the known discharge, friction slope, and the hydraulic properties of the flume. The Manning's n-value of the flume was determined to be about 0.009 based on clear-water flows. The Manning's n-value of the prototype concrete-lined channel was assumed to be 0.014 with clear-water flows (neglecting side-wall effects).

Scaling of the increase in hydraulic roughness between the flume and the prototype was based on the following. The total hydraulic roughness can be expressed as:

$$n_{total} = n_{bed} + n_{bedload}$$

where

n_{total} = total roughness due to bed and bedload roughness,
n_{bed} = roughness due to the bed of the channel, and
$n_{bedload}$ = roughness due to bedload movement.

Solving for the roughness due to bedload movement, the above equation can be written:

$$n_{bedload} = n_{total} - n_{bed}$$

The model ratio of 1:32.1 is used to convert the $n_{bedload}$ from the flume to the prototype as follows:

$$n_r = L_r^{1/6} = \left(\frac{1}{32.1}\right)^{1/6} = \frac{1}{1.783}$$

or

$$n_{p_{bedload}} = 1.783\, n_{m_{bedload}}$$

where

n_r = model ratio for Manning's n-value = n_m/n_p,
n_p = n-value of prototype,
n_m = n-value of model, and
L_r = model ratio (equals 1 model to 32.1 prototype for this study).

Hence, the total hydraulic roughness neglecting sidewall effects and including bedload movement in the prototype is:

$$n_{p_{total}} = 0.014 + 1.783 n_{m_{bedload}}$$

Flume Results. The tests were conducted using two uniform grain sizes and a gradation that simulates the gravel portion of the creek bed. The uniform grain sizes simulate the prototype d_{84} (108 mm, σ = 1.4) and the large boulders found in the armor layer upstream from the proposed project, d_{max}, (216 mm, σ = 1.4). The concentrations ranged up to 5000 ppm, which was considered sufficiently high to simulate the gravel bed load in Mission Creek. Actual prototype gravel bedload concentrations were determined by the numerical model study.

Results from the flume tests are still preliminary in nature and are shown in Table 2 and Figure 3. Tests for the d_{max} material were conducted with concentrations varying from 200 to 5000 ppm. The gravel bed material was observed to move down the flume in bouncing or rolling motions at velocities slightly less than the water. Between 200 and 800 ppm, there was a minor increase in roughness from 1.4 to 5.7 percent. Between 1600 and 3000 ppm, the Manning's roughness coefficient increased from 8.6 to about 16.4 percent. And at 3250 ppm, the flume tests indicated that the flow regime becomes unstable and bed forms began to develop.

Tests for the d_{84} material were conducted with concentrations varying from 200 to 3000 ppm. Similar to above, the gravel bed material moved down the flume in bouncing or rolling motions at velocities slightly less than the water. Between 200 and 800 ppm, there was a minor increase in roughness from 1.4 to 6.4 percent. Between 1600 and 3000 ppm, the Manning's roughness coefficient increased from 8.6 to about 16.4 percent. For the d_{84} material, the flow regime became unstable at about 3000 ppm.

Table 2. Effect of bed-load movement on hydraulic roughness

Sediment Size	Bedload Concentration (ppm)	$n_{bedload}$ (prototype)	% Increase
d_{max}	0	0.0140	0.0
	200	0.0142	1.4
	400	0.0143	2.1
	800	0.0148	5.7
	1600	0.0152	8.6
	2400	0.0159	13.6
	3000	0.0163	16.4
	3250	0.0166	18.6
d_{84}	0	0.0140	0.0
	200	0.0142	1.4
	400	0.0143	2.1
	800	0.0149	6.4
	1600	0.0152	8.6
	2400	0.0159	13.6
	2700	0.0162	15.7
	3000	0.0163	16.4
Bed Gradation	0	0.0140	0.0
	500	0.0143	2.1
	1000	0.0149	6.4
	1500	0.0151	7.9
	2000	0.0157	12.1
	3000	0.0160	14.3

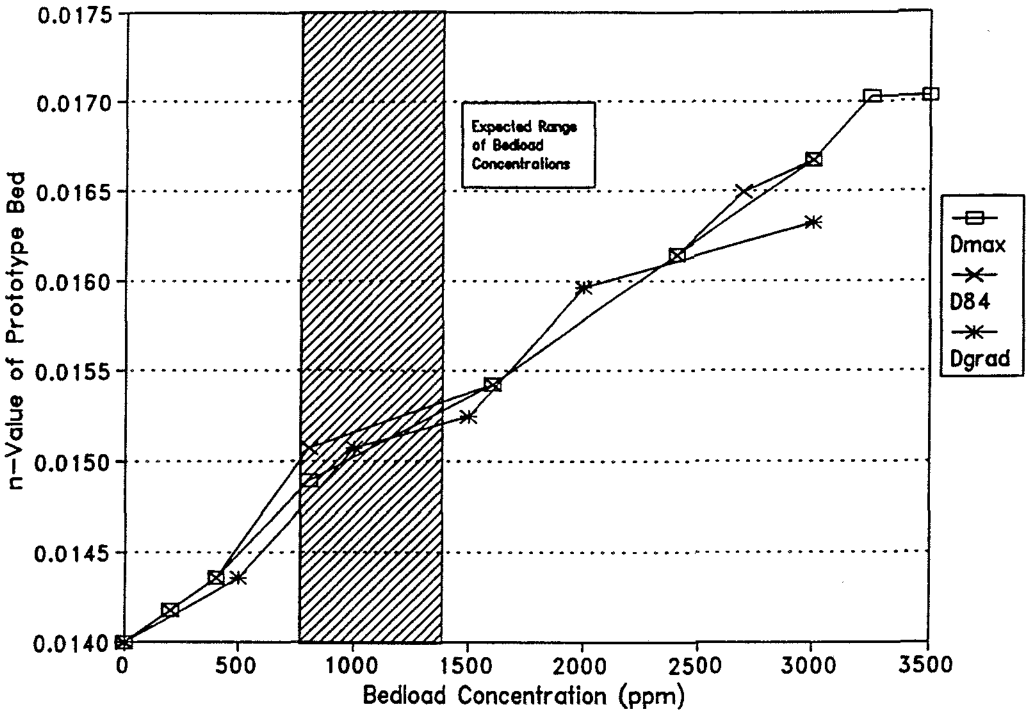


Figure 3. Effect of bed-load transport on hydraulic roughness

Tests for the bed material gradation were conducted with concentrations varying from 500 to 3000 ppm. Between 200 and 1500 ppm, there was a minor increase in roughness from 2.1 to 7.9 percent. Between 2000 and 3000 ppm, the Manning's roughness coefficient increased from 12.1 to about 14.3 percent. A maximum threshold concentration for the bed material gradation was not determined.

Review of the flume data reveals two points: 1) the results are not very sensitive to the grain size for the hydraulic conditions and grain sizes modelled; and 2) the flow regime will be supercritical for the anticipated range of bedload concentrations.

CONCLUSIONS

Conclusions are preliminary at this time. Predicted bed-load transport rates from the numerical model have been coupled with measured hydraulic roughnesses from the flume study to determine the effect of the gravel bed load for the Mission Creek Project. Based on the numerical modeling completed by the Los Angeles District, the maximum concentration of bedload material inflowing the proposed concrete-lined channel will be about 1300 ppm. Preliminary results from the flume study indicate that this maximum bedload concentration will increase the maximum hydraulic roughness from 0.014 to 0.0153. Additionally, results from the flume study reveal that bed forms do not form under project conditions for concentrations up to 3000 ppm.

APPENDIX I. CONVERSION FACTORS, UNITS OF MEASUREMENT

To Convert	To	Multiply By
inch (in)	centimetre (cm)	2.54
foot (ft)	metre (m)	0.305
mile (mi)	kilometre (km)	1.6
square mile (mi^2)	square kilometre (km^2)	2.59
cubic foot per second (cfs)	cubic metre per second (m^3/s)	0.0283

APPENDIX II. REFERENCES

Simons, Li & Associates, Inc. (1984). "Debris deposition study for without-project and with-project conditions, Santa Barbara County streams, Mission Creek/Rattlesnake Creek, Santa Barbara County."

US Army Engineer District, Los Angeles (1986). "Feasibility report and environmental impact statement, Lower Mission Creek interim, Santa Barbara County streams, California."

Vanoni, V.A. (1964). "Transport of suspended sediment by water." Transactions, ASCE, paper 2267, 111:129-160.

RIO PUERTO NUEVO SEDIMENTATION STUDY

by

[1]Eric Holand, [2]Brad Hall

INTRODUCTION

Puerto Rico is a beautiful island 500 miles from southeast of Florida in the Caribbean Sea. The island is typically comprised of a mountainous interior which slopes sharply to a coastal plain. The largest city in Puerto Rico is San Juan. This city is not a rapidly developing major metropolitan area. Economic growth has increased the demand and value of available land. This has often resulted in construction close to major floodways. The basin is subject to runoff discharges typical of hurricane and tropical storm rainfall on steep slopes. The Corps of Engineers was charged with developing plans for providing flood control for the Metropolitan San Juan Area.

Objective of the Sediment Study

The objective of the sediment study was to determine the size of sediment control features for the proposed project channels. Flood control features planned would incorporate the use of supercritical concrete lined channels. A supercritical design was chosen due to the steep slopes of the existing river channels, the high cost of right-of-way, and the desire to avoid the socio-economic impacts of rebuilding several bridges of the heavily utilized expressway system. Flow conditions within the project channel would produce average velocities up to 30 feet per second. Erosion of concrete surfaces would be expected unless transport of gravel and sand particles can be minimized. Sand and gravel size particles entering project channels could also develop into bedforms which would alter roughness coefficients which would adversely impact design flood control stages. Debris basins at the upstream end of the project channels for the Rio Piedras and Quebrada Guaracanal were chosen as sediment control features.

Basin Characteristics

Figure F-1 shows a location map of the Rio Piedras basin for which project flood control features are proposed. The entire basin is either fully developed or is under intense pressure for development by residential housing, commercial, industrial and transportation interests. The basin was considered to be 75% developed in 1980. Current development trends indicate that the entire basin will be 80% developed for residential or commercial uses by the year 2000.

The basin may be divided into three areas according to slope changes. The coastal area ranges from the Bay of San Juan to the Buena Vista tributary of Rio Piedras. That area has an average slope of 0.0017 ft/ft. Ground cover

[1]Chief, Hydraulic Data and Design Section, Hydrology and Hydraulics Branch, Jacksonville District, U.S. Army Corps of Engineers

[2]Research Hydraulic Engineer, Hydraulics Laboratory, Waterways Experiment Station, U.S. Army Corps of Engineers

consists of closely spaced residential and commercial buildings, paved roads and parking facilities. Rio Piedras discharges into the Puerto Nuevo channel which then discharges to San Juan Harbor.

The second area from Buena Vista to the Winston Churchill Avenue Bridge (PR-177) has an average slope of 0.004 ft/ft. The area north (downstream) of the Winston Churchill Avenue bridge is also fully developed and runoff is typical for urban areas. Local drainage ditch and culvert systems quickly deliver runoff to Rio Piedras and Rio Puerto Neuvo.

The basin upstream of the Winston Churchill Avenue bridge is wooded and mountainous but is rapidly being developed for urban and residential purposes. Roads are being upgraded and new expressway systems are being constructed. Average channel slopes range from 0.004 ft/ft initially to 0.006 ft/ft in upstream areas. Vegetative cover is tropical and recovers quickly from disturbance.

Hydrologic conditions

Rainfall in the basin averages 75.4 inches per year. The climate is tropical with a relatively small variation in temperature. Flood events are characterized by intense rainfall with large volumes of runoff occurring within a few hours. The conveying river or stream may reach and exceed bankfull within hours after the initial rainfall and return to pre-event flows within a 24-hour period. The upper basin, south of the Winston Churchill Avenue bridge, is characterized by steep slopes and soil conditions which provide only minimal opportunity for infiltration.

Existing Flood Channel Features

The primary existing drainage feature of this basin is Rio Piedras. This river has its headwaters in the mountainous upper basin area and discharges into the channelized reach of Rio Puerto Nuevo near the De Diego Avenue bridge. Rio Puerto Nuevo then discharges into San Juan Harbor. One smaller tributary, Quebrada Margarita, also discharges into Rio Puerto Nuevo.

a. Rio Piedras. Rio Piedras is uncontrolled and overflows into adjacent developed areas during major flood events. The existing channel seems to be relatively stable with deposition and scour in balance. Stream banks are very steep to near vertical with some areas showing moderate bank erosion. Available rock in the channel of the upper basin stream bed provides armoring against scour.

b. Tributaries to Rio Piedras. Local runoff from streets, houses and commercial buildings in the area south of Winston Churchill Avenue Bridge drain to tributaries which convey flow to Rio Piedras. Three major tributaries are large enough to require open channel discharges into Rio Piedras. Those tributaries (Quebrada Guaracanal, Quebrada Josefina/Dona Anna, and Quebrada Buena Vista) are concrete lined. Runoff enters the tributaries through standard street culvert systems and earth lined ditches at the upstream end of each channel. Discharges from the remaining tributary areas are small enough to enter the main channel directly through street culverts.

c. Quebrada Margarita. The existing upper basin channel is trapezoidal in shape and lined with concrete. Development has reached the banks along the entire length of the channel right-of-way. Runoff from the surrounding area enters the channel through culverts or inlet structures.

Problem Identification

a. Sediment Sources. Land clearing activities associated with increased development in the upper basin area were determined to be the prime source of existing sediment loads being transported in Rio Piedras. Sediment and debris production from this reach of the basin will continue until development of the basin reaches completion and construction activities subside. Under existing conditions, floods which exceeded the channel banks deposit some sediments in areas outside the main channel. The proposed channelization of the Rio Piedras would transport most of the sediment load entering the channels to San Juan Harbor. Maintenance dredging cost the Puerto Nuevo Channel could increase unless those sediments are reduced.

An inspection of existing concrete lined channel reaches of tributaries revealed the existence of small sandbars within the existing channels. In some cases, vegetation (small trees, brush, grasses, etc.) growing on the sand bars indicated that they are relatively stable and have been in existence for some time. Water in most of the channels appeared to be clear with very little wash load noted. Typically, a section of the concrete channel lining upstream of sandbars had failed and was experiencing loss of backfill material into the channel. Since the areas upstream are completely developed, the sand bars were probably formed from backfill material from those failure sites.

The soils within the upper basin contain grain sizes varying from silts and clays to gravels, large cobbles and boulders. The existing channel in the undeveloped areas upstream of the proposed project channel passes through areas which have large rock formations. Those areas are frequently armored with boulders which provide little or no possibility for scour. Rock formations have smooth surfaces indicating the degree of polishing due to sands transported during flood events. Baseflows observed in unlined channels ranged from clearwater to a small washload of silts and clays.

Sand and gravel bar formations in the project reaches of the existing channel were also noted in channel reaches downstream of the debris basin site where land slopes become moderate. These bars consist of medium to fine gravels and course sands. An absence of clay, silts and sand fines was noted, but this is not considered unusual due to the slope of the existing channels and the turbulent and high velocities experienced during discharge events.

Soils in areas adjacent to San Juan Harbor have higher contents of clay and silt. Dredging records of San Juan Harbor show large percentages of fine silt and clay particle sizes.

b. Floating Debris. Field Reconnaissance noted accumulations of debris against bridge piers at several bridge locations. The majority of the debris observed consisted of vegetation and wood building materials. This causes a constriction of flow and increase flooding due to backwater effects.

c. <u>Sediment Rates for Adjacent Areas</u>. Sedimentation rates for various reservoirs in Puerto Rico range from as low as 0.86 af/yr/sq.mi. to as high as 7.8 af/yr/sq.mi. The sediment producing drainage basin area upstream of the proposed site of the debris basin is 8.29 sq. miles. The high level of development being experienced in the areas above the project would indicate that use of a high sediment production rate would be justified for design. The following computations show sediment production expected at Rio Piedras Debris Basin by using the highest measured rate of sediment load for a regional reservoir excluding clay and silt:

7.8 af/yr/sq.mi. x 8.29 sq.mi. - 64.66 af/year

64.66 af/year x 30% (suspended sand)	-	19.4 af-ft
64.66 af/year x 10% (bedload)	-	1.94 af-ft
Total		21.34 af-ft/ year

<u>Study Approach</u>

<u>Sediment Data Collection</u>. Suspended sediment monitoring stations were set-up and operated by the United States Geological Survey (USGS) at four locations on the Rio Piedras. Those stations have been operational since 1988. Data collected were analyzed and log-log plots to show suspended sediment discharge in tons per day, concentration in mg/l and percent sand to water discharge were compiled.

a. Station 50048770 was established at the proposed site of the Rio Piedras debris basin at the Winston Churchill Avenue bridge. This site was chosen because it is at the most upstream end of the proposed project concrete channels. Data from this area showed suspended load from undeveloped areas and the natural stream. Figures F-2, F-3, and F-4 show the data collected.

b. Station 50049000 was established at PR-1 bridge over Rio Piedras. A comparison of sediment data from this station and Station 50048770 assisted in forming a determination of the level of sediment loading contributed by Quebrada Guaracanal.

c. Station 50049310 was established at the J.T. Pinero Avenue bridge over Quebrada Josefina. This location was chosen to determine the sediment load being contributed by Quebrada Dona Anna and Quebrada Josefina. The Dona Anna and Josefina channel segments are lined with concrete for most of the basin length and provide drainage for highly developed urban areas. They are typical of tributaries from developed areas which discharge into Rio Piedras.

d. Station 50049100 was established at the J.T. Pinero Avenue bridge over Rio Piedras. This station is located at the midpoint of the proposed channels and was provided to assist in determining the volume and consistency of sediment contributed by the Buena Vista tributary. A large sand and gravel bar has formed in the area downstream of the confluence of that tributary and the main channel of Rio Piedras.

Of primary importance for the sediment study are stations on Rio Piedras at USGS Gage No. 50048770, and USGS Gage No. 50049000. The Rio Piedras at USGS

Gage No. 50048770 is located at the site of the proposed debris basin near Winston Churchill Avenue. The Rio Piedras at Rio Piedras gaging station is located downstream of the confluence of Quebrada Josefina with Rio Piedras.

Suspended Sediment Relationships.

a. Data Comparisons. A comparison of data between stations 50048770, 50049000, and 50049310 is shown by Figure F-5. This figure shows a decline in the sediment concentration versus discharge. Station 50049000 was established to determine the necessity of providing a debris basin at Guaracanal and Rio Piedras. The decline in concentration of sediment shows that very little, if any, sediments were being transported to Rio Piedras from that basin at the time of sampling. To determine if there is a significant difference between the total population of sediment samples, which includes storm event samples, baseflow samples, and individual storm event sediment production this report focused on the sediment samples taken during individual storm events. Observed suspended sediment concentration versus flowrate sampled during individual storm runoff events were plotted and compared. Comparison of concentrations recorded for Station 50049100 and Station 50049000 shows a further declining sediment load as flow moves downstream to San Juan Harbor. This would indicate that flows from Buena Vista Tributary are not contributing appreciable sediment loads to Rio Piedras. The Buena Vista tributary is served by a concrete channel and is typical of highly developed areas tributaries to Rio Piedras.

b. Coincident Storm Analysis. Flowrate and concentration was simultaneously sampled at station 50048770 and 50049000 on August 24, 1988 and April 18, 1989. These two storms provided insight into the relative contribution of runoff and sediment discharge between the Rio Piedras and Guaracanal watersheds. The sampled hydrographs and sediment concentrations indicated that both peak discharge and runoff volume on the Rio Piedras are higher downstream of Quebrada Guaracanal, however sediment concentrations are much lower at this point on Rio Piedras than that which is sampled upstream of the confluence of Quebrada Guaracanal. This indicates that runoff from Quebrada Guaracanal does not carry a high sediment load during storm events.

c. Regression Relationships. Unit discharge relationships for the sampled suspended sediment load at stations 50049310, 50049000 and 50048770 were developed. This provided an additional means to estimate the significant sediment inflow from Quebrada Guaracanal. The unit discharge-suspended sediment concentration relationships are plotted on Figure F-5. The suspended sediment concentration-unit discharge regression indicates higher concentrations at gage number 50048770. In the absence of tributary inflow this decrease in unit discharge versus concentration is expected between gages 50049000 and 50048770, since the watershed sediment delivery ratio decreases with increase in watershed area. Since the Guaracanal is a major tributary inflow point between the two gages, the reduction in concentration versus unit discharge can be attributed to both reduction in sediment delivery ration due to sediment storage in the Rio Piedras channel and proportionally reduced sediment inflow from the Guaracanal.

d. Estimate of Percent Wash Load. The percent of the sampled suspended load finer and coarser than 0.0625 millimeters was determined for a few of the suspended load samples. These samples are useful for determining the suspended sediment load that comes from bed material transport (sand load) and the portion of the suspended load that comes from watershed erosion and is primarily carried as wash load (silt and clay load). Regressions of the percent sand in the suspended sediment samples were regressed with both the total suspended sediment concentration and with the flowrate. The results indicate that the percent washload is independent of the flowrate and sediment concentration for station 50048770 and that there is a slight increase in the percent suspended sand for station 50049000. About 30 percent of the suspended load sampled at station 50048770 is sand.

Figure F-6 indicates that the percent sand for station 50048770 is nearly independent of discharge and concentration for the range of samples taken. Sand constitutes about 30 percent of the sampled suspended sediment for this station. The percent sand at station 50049000 indicates a slight increase in the percent sand with an increase in discharge.

Figure F-7 indicates which of the flowrate-suspended sediment concentration data points that the percent sand in the sample was determined. The plot indicates that several of the data points for station 50048770 were sampled from the highest sampled discharges. The slope of the regression line for the percent sand data points is nearly identical to the slope for all data points for both stations 50048770 and 50049000.

e. Suspended Sediment Particle Size Distribution. The ratio of grain sizes was determined by computing a weighted mean of standard size gradations from sediment samples taken from USGS sampling stations. The corresponding percents of each grain size were applied to the suspended sediment rating curve. A weighted mean average of sediment discharges and corresponding percent finer grain size analyses was compiled of all the samples taken. The computed average of the samples was 67 percent finer than very fine sand (VFS) size particles. Therefore, about 33 percent of an average sample would be comprised of sand size particles. This compares favorably with the percent sand versus concentration analyses.

Sediment Inflow Characteristics

a. Sediment Concentration Relationship. The regression relationships for concentration versus flowrate measured for individual storm events on the Rio Piedras and Quebrada Josefina were developed for determining design inflow quantities for formulating the upper range of the sediment discharge relationship. The relationships were used to the design of sediment basin design. The flowrate-concentration regression plotted in Figure F-5 for station 50048770 and station 50049310 are:

Rio Piedras $C_s = 10{,}800 \quad Q^{0.11}$

Quebrada Josefina $C_s = 152 \quad Q^{0.243}$

where C_s is the total suspended sediment concentration in mg/liter, and Q is the flowrate in cfs.

b. Sediment Transport Function. The Larsen Madden transport function was chosen for use in HEC-6 modeling of the Rio Piedras debris basin. This transport function was recommended after a review of the field data.

c. Bed Material. Figures F-8, F-9 and F-10 show typical grain size analyses of samples taken from selected cross sections used in computer modeling. Site investigation also revealed that the channels and side slopes contained rock ranging from 50 mm (2 inches) to 650 mm (2+ feet). Large rock was noted to comprise from 8% to 25% of the sample. This condition would result in relatively fast armoring of the streambed as discharges increase.

Debris Basin Design

Basic Design Criteria. The flow regime for the project area is characterized by large runoff events with short (less than 24 hour) durations which are preceded and followed by small average daily discharges. Base flow is very small and has negligible sediment carrying capability. Single storm events were determined to provide the best estimation of the required capacity of a debris basin. Since the protection frequency for the design channel was chosen as the 1-in-100 year frequency, the debris basin was also sized to trap potentially damaging sediment load from a 1-in-100 year event. Silt and clay particle sizes are not considered highly abrasive and capture of sediment particles finer than very fine sand would require a reservoir size basin. In addition, high design velocities in the project channel would allow only minimal amounts of clay and fine silt particles to settle out of suspension.

Computer Analyses. The U.S. Army Hydrologic Engineering Center computer program, HEC-6, "Scour and Deposition in Rivers and Reservoirs" was utilized to compile a computer model for the proposed project sediment basin and the existing channel upstream of Winston Churchill Avenue Bridge and Quebrada Guaracanal. Suspended sediments and bed load movement from the areas upstream of, and tributary to, the project channels were modeled for single storm hydrographs and flow duration hydrographs were formulated to provide a input for HEC-6 modeling of trial design debris basins.

Program Input Data. Survey cross sections of the existing Rio Piedras channel upstream of the proposed project channels and samples of the bed material were taken and analyzed for grain size composition. Twenty eight cross sections were surveyed and used to formulate a sediment transport model using HEC-6. Sediment discharge rating curves were developed for a range of discharges which included the peak discharge of the SPF flood. Sediment basins were modeled as if they were reservoirs. Discharges are high and time to peaks are small. Time steps as small a 2 minutes were used for high discharges. This provided a smooth deposition profile of sediments in the basin.

Channel Roughness. Natural channel reaches upstream of the debris basin are comprised of rock and gravel armored bottoms with heavily vegetated side slopes. Channel bottoms are also characterized by bedforms and reaches which have large immovable rock formations. Manning's roughness coefficients of 0.1 was used for left and right bank areas. The main channel was modeled with a roughness coefficient of 0.06. A roughness coefficient sensitivity analyses showed no change in trap efficiency or volume of sediments moved to the debris basin when roughness coefficients were varied.

Conclusions.

Puerto Nuevo Debris Basin. Figure F-11 shows a plan view of the debris basin and outlet structure. The basin would be constructed in a reach of the natural channel by excavating the east bank to provide a "tear drop" shape. Total volume under the elevation of the top of the outlet structure weir is 104 acre feet. Basin area is about 9.65 acres. The bottom elevation of the basin was set at the elevation of the existing streambed. This would minimize head cutting potential.

The model starting point was set at the debris basin outlet structure and extends for a distance of 3,580 feet upstream. Hydrographs of the 10-year, 25-year, 50-year, 100-year and SPF were then run through the basin. Results were analyzed to determine the appropriate time steps for each discharge which would produce smooth sediment deposition or degradation in the model. Table T-1 shows the performance of the basin under the 1-in-10 year through the SPF storm hydrographs. Figure F-12 shows profile plots of the results of sediment settlement in the debris basin for each frequency.

Flow duration analyses were also performed by running a hydrograph based on a flow duration hydrograph which simulated consecutive years of discharges without cleaning and reshaping the basin to design specification. Table T-2 shows the expected decrease in trap efficiency over a three year period of time. Figure F-13 shows profile plots of the results of sediment settlement in the debris basin for three years of continuous flows without annual cleaning and reshaping. Maintenance clearing and reshaping should be planned on an as needed basis, especially after severe runoff events. Monitoring of basin capacity should be scheduled on a three month basis to insure adequate capacity.

Guaracanal Debris Basin. Although sediment analyses showed that only small sediment loads were probably being discharged from this basin at the time of sampling, a full size basin was considered prudent. An analyses similar to that for the Rio Piedras Basin was performed.

Tributaries. Sediment production from Quebrada Margarita, Quebrada Dona Anna, Quebrada Josefina, Quebrada Buena Vista and Quebrada Guaracanal were found to be minimal due to the developed nature of the area. Analyses of suspended sediment samples taken by field sampling showed that concentrations of sediment from the open channel tributaries were very low. Sediment loads for the 10-year through SPF hydrographs were computed and shown to provide minimal sediment to the project channels. Sediment samples showed little sand content. Sediment loads would consist of silt and clay and some fine sand grain sizes which would not deposit in appreciable quantities due to the high velocity nature of the flow regime.

Total Yearly Sediment Load. Flow duration analyses of the main channel and tributaries were compiled to show the expected sediment loading for one year. Table T-3 shows a summary of those loads. Total sediment load to San Juan Harbor could increase because of loss of some overbank flood areas. However, the existing channel of Rio Piedras is well defined and urban development, which has encroached up to the banks of Rio Piedras, affords little opportunity for extended periods of overbank flooding which would result in deposition of clay and silt particles. However, the project channel is not expected to appreciably increase sediment transport to San Juan Harbor.

Floating Debris. Capture of floating debris is considered to be impractical. Existing bridge pier bents would be furnished with sloped nose extensions and diaphragm walls to deflect floating debris and minimize debris buildup against pier groups. Analyses of design channels were based on the assumption that floating debris would build up against bridge piers for a thickness of two feet on each side of each pier group.

Comparison of Regional and Computed Sediment Loads.

Table T-3 shows a summary of post project sediment loads based on the sediment rates used for design of the debris basins. Those rates would produce an average yearly sediment load to San Juan Harbor of 272 Acre feet. This compares favorably with dredging records for San Juan Harbor which show an average of 158 af/year. Only the ship channels are dredged. However, those channels comprise a large part of the harbor and are located directly along the flow path of discharges from Rio Puerto Nuevo outlet. Therefore, the analyses is a considered a conservative projected figure when compared to dredging records and yearly reservoir loads for Puerto Rico.

The current rate of sediment transport being experienced in this basin is elevated due to the current rate of development in the project basin. Debris basins are subject to wet years which produce larger quantities of sediments and should be designed for those higher load conditions. Larger basins would also serve to reduce overall maintenance clearing requirements.

Table T-1
Rio Puerto Nuevo
Winston Churchill Avenue Debris Basin
Summary of Trap Efficiency

Frequency of Storm (Year)	Peak Discharge cfs	Sand/Gravel Inflow Ac ft	Sand/Gravel Outflow Ac ft	Trap Efficiency %
10	18,680	14.17	0.02	99.9
25	21,600	17.82	0.05	99.7
50	23,100	20.08	0.07	99.7
100	24,500	22.13	0.15	99.3
SPF	27,700	49.94	5.03	89.9

Note: Clay and Silt size particles would not be trapped

Table T-2
Rio Puerto Nuevo
Winston Churchill Avenue Debris Basin
Summary of Trap Efficiency for
Flow Duration Analyses

Years of Operation	Peak Discharge cfs	Gross Sand/Gravel Inflow Ac ft	Gross Sand/Gravel Outflow Ac ft	Trap Efficiency Gross %	Trap Efficiency Yearly %
1	18,392	50.1	0.02	99.8	99.8
2	18,392	100.0	17.86	89.0	64.2
3	18,392	149.9	42.00	71.0	51.6

Note: Assumes no clearing of sediments during interval of filling

Table T-3
Rio Puerto Nuevo
Projected Annual Sediment Load
From Tributaries and Main Channel
in Acre Feet

Basin		Sand/Gravel	Clay-Silt	Total
Rio Piedras	(1)	-	174.60	174.60
Guaracanal	(1)	-	90.70	90.70
Buena Vista		0.18	0.99	1.17
Josefina		0.38	2.02	2.40
Dona Anna		0.23	1.24	1.47
Margarita		0.32	1.69	2.01
Sum		1.11	271.24	272.35

(1) Note: Sand and gravel sizes captured by Debris Basins

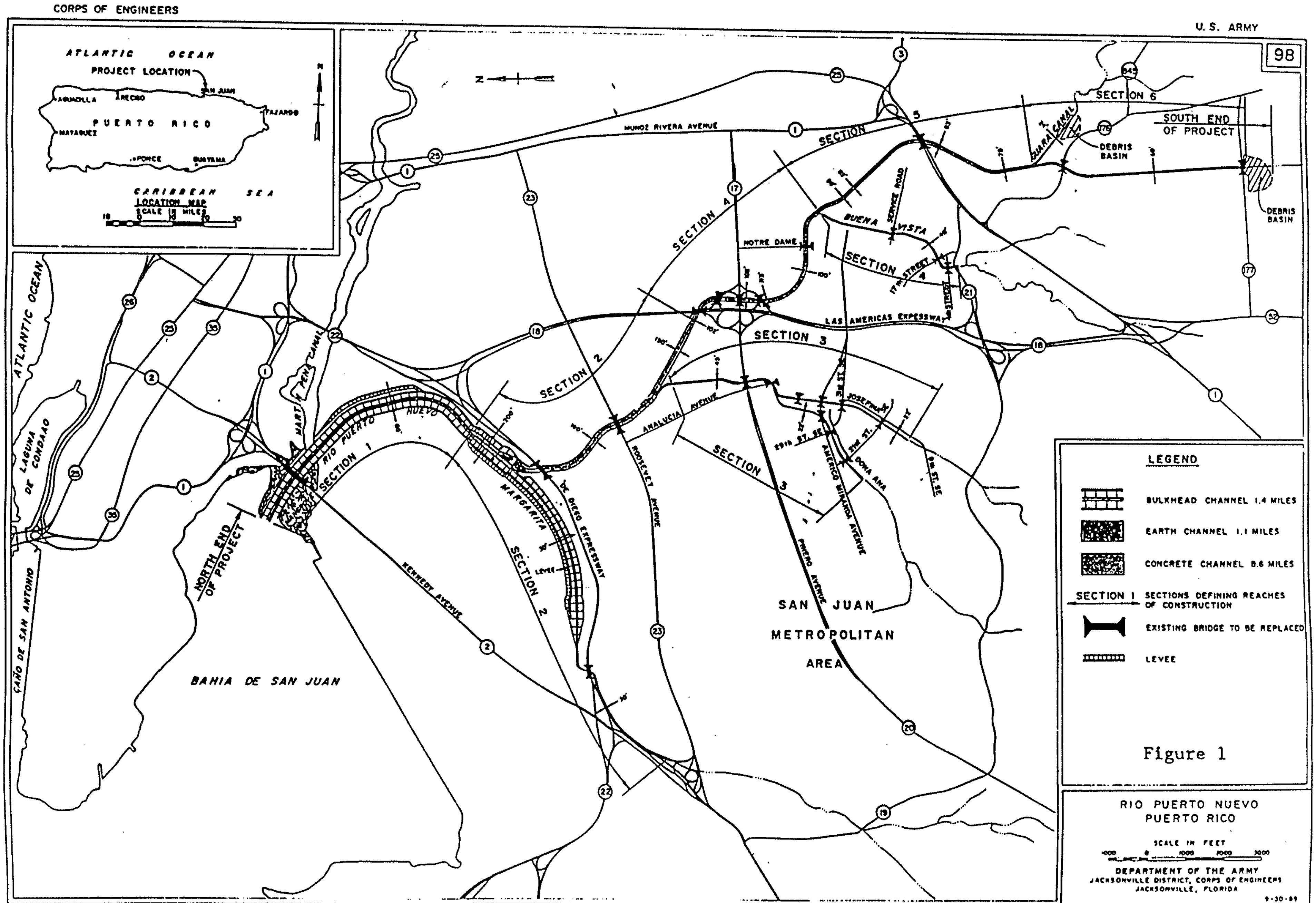

CORPS OF ENGINEERS
U. S. ARMY
98
ATLANTIC OCEAN
PROJECT LOCATION
SAN JUAN
AGUADILLA
ARECIBO
FAJARDO
MAYAGUEZ
PUERTO RICO
PONCE
GUAYAMA
CARIBBEAN SEA
LOCATION MAP
SCALE IN MILES
ATLANTIC OCEAN
LAGUNA DE CONDADO
CAÑO DE SAN ANTONIO
BAHIA DE SAN JUAN
NORTH END OF PROJECT
SOUTH END OF PROJECT
MUNOZ RIVERA AVENUE
MARTIN PEÑA CANAL
RIO PUERTO NUEVO
SECTION 1
SECTION 2
SECTION 3
SECTION 4
SECTION 5
SECTION 6
KENNEDY AVENUE
MARGARITA
DE DIEGO EXPRESSWAY
LEVEE
ROOSEVELT AVENUE
ANALUCIA AVENUE
LAS AMERICAS EXPRESSWAY
NOTRE DAME
BUENA VISTA
SERVICE ROAD
17th STREET
JOSEFINA
29th ST. SE
AMERICO MIRANDA AVENUE
DOÑA ANA
PINERO AVENUE
GUARACANAL
DEBRIS BASIN
SAN JUAN METROPOLITAN AREA
LEGEND
BULKHEAD CHANNEL 1.4 MILES
EARTH CHANNEL 1.1 MILES
CONCRETE CHANNEL 8.6 MILES
SECTION 1 SECTIONS DEFINING REACHES OF CONSTRUCTION
EXISTING BRIDGE TO BE REPLACED
LEVEE
RIO PUERTO NUEVO
PUERTO RICO
SCALE IN FEET
DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS
JACKSONVILLE, FLORIDA

Figure 1

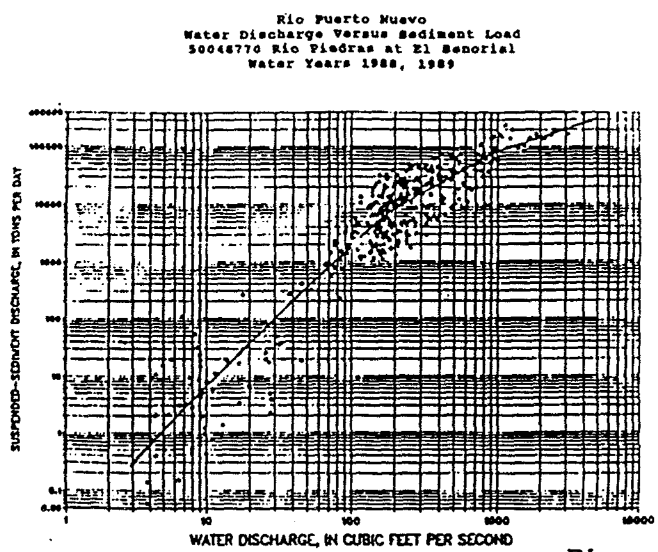


Figure F-2

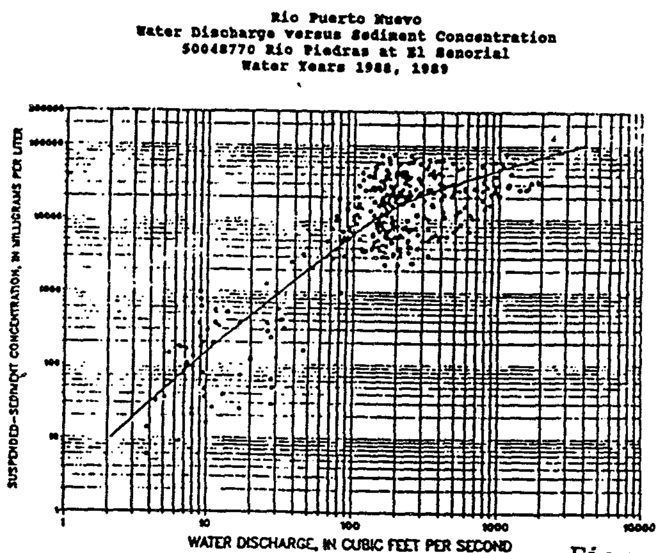


Figure F-3

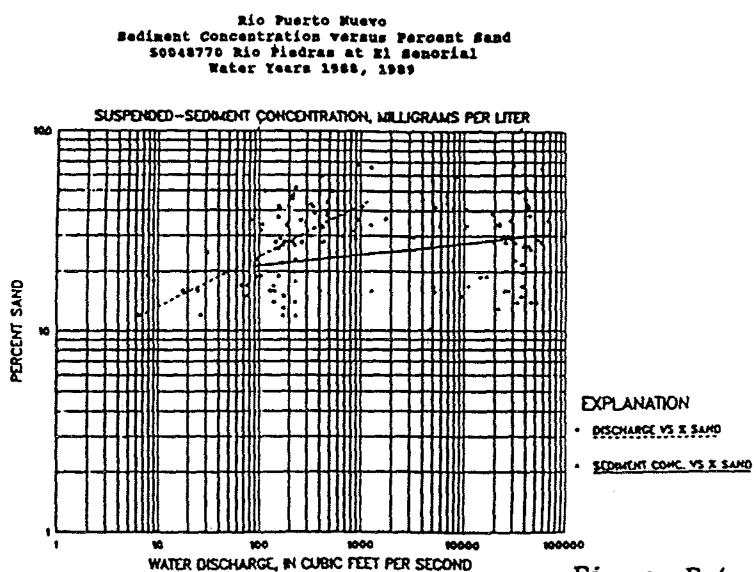


Figure F-4

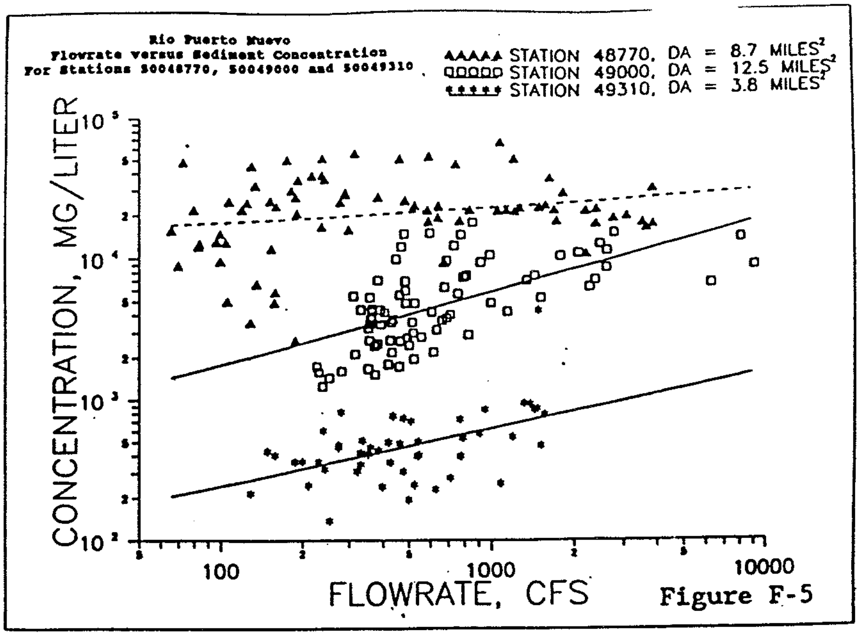


Figure F-5

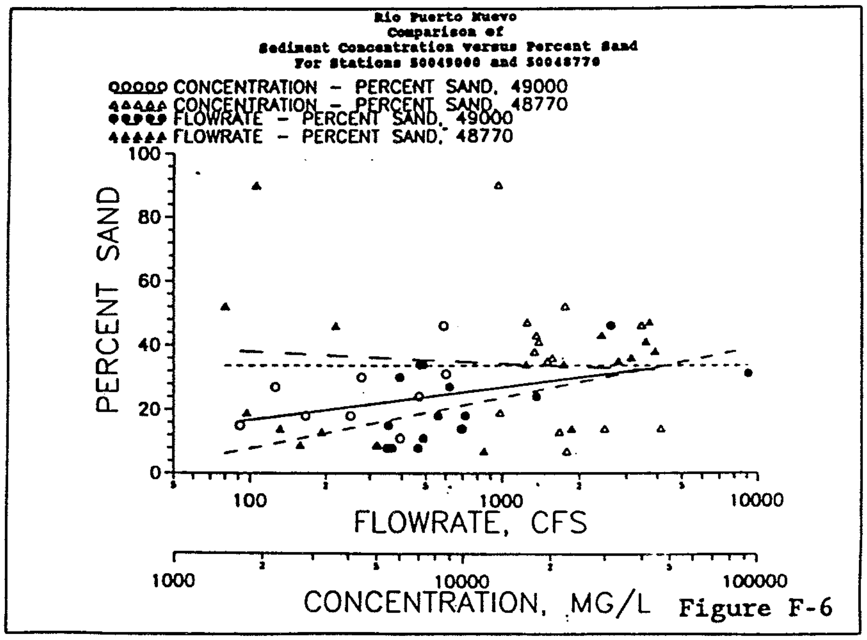


Figure F-6

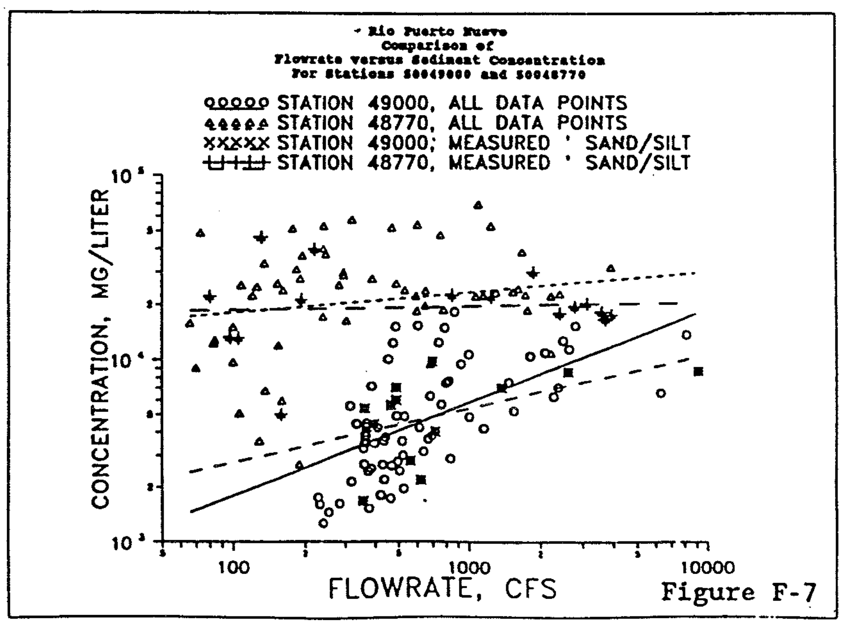


Figure F-7

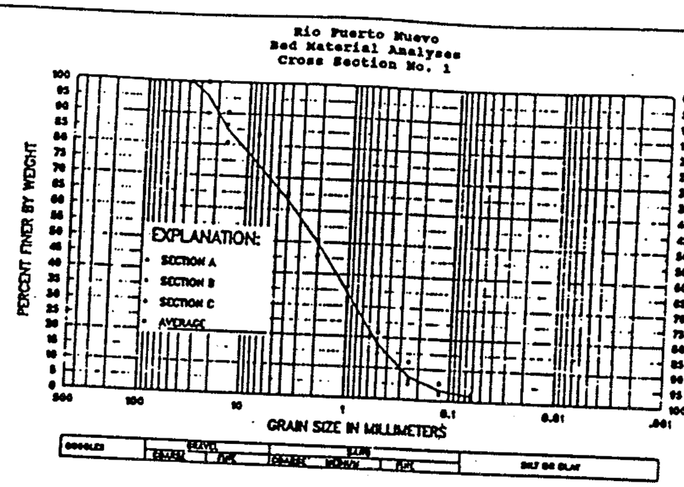


Figure F-8

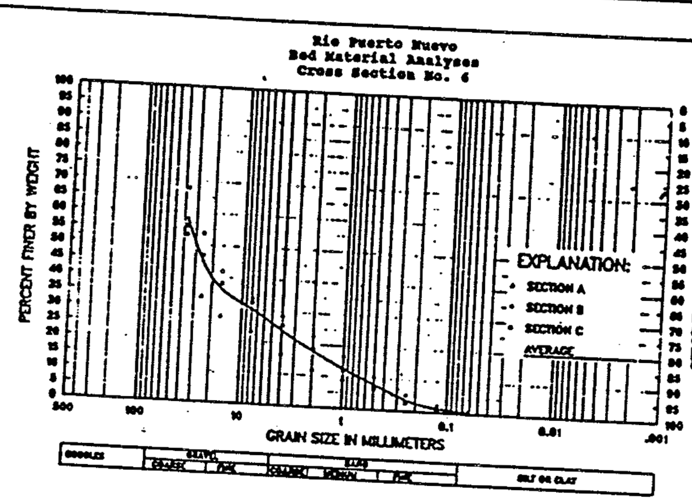


Figure F-9

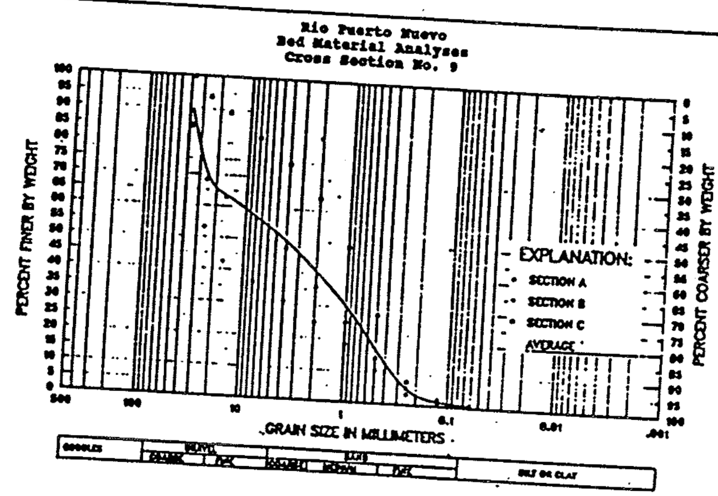
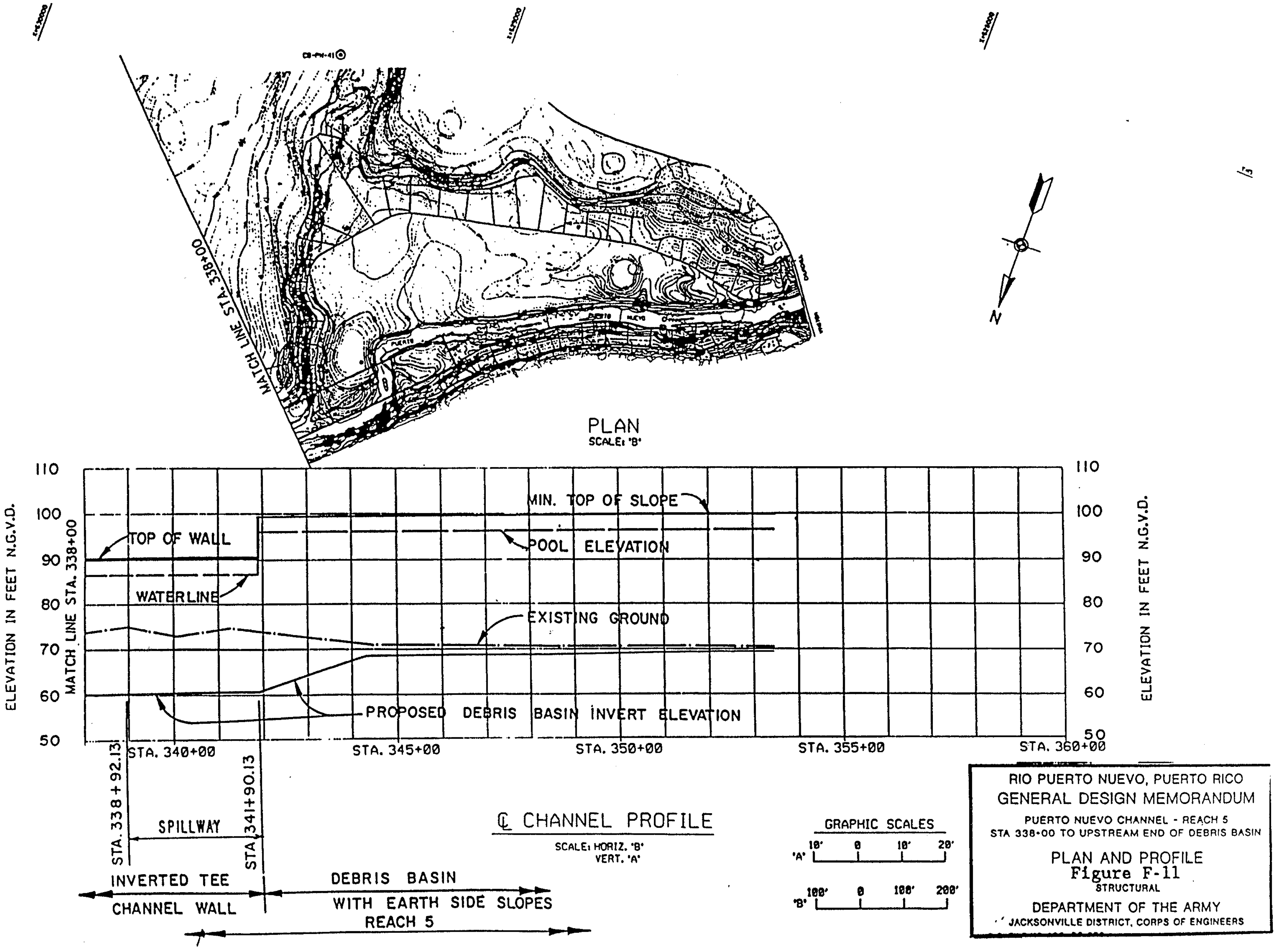


Figure F-10



PLAN
SCALE: 'B'
MATCH LINE STA. 338+00
N
110
100
90
80
70
60
50
ELEVATION IN FEET N.G.V.D.
MATCH LINE STA. 338+00
MIN. TOP OF SLOPE
TOP OF WALL
POOL ELEVATION
WATERLINE
EXISTING GROUND
PROPOSED DEBRIS BASIN INVERT ELEVATION
STA. 340+00
STA. 345+00
STA. 350+00
STA. 355+00
STA. 360+00
STA. 338+92.13
STA. 341+90.13
SPILLWAY
INVERTED TEE
CHANNEL WALL
DEBRIS BASIN
WITH EARTH SIDE SLOPES
REACH 5
℄ CHANNEL PROFILE
SCALE: HORIZ. 'B'
VERT. 'A'
GRAPHIC SCALES
'A' 10' 0 10' 20'
'B' 100' 0 100' 200'
RIO PUERTO NUEVO, PUERTO RICO
GENERAL DESIGN MEMORANDUM
PUERTO NUEVO CHANNEL - REACH 5
STA 338+00 TO UPSTREAM END OF DEBRIS BASIN
PLAN AND PROFILE
Figure F-11
STRUCTURAL
DEPARTMENT OF THE ARMY
JACKSONVILLE DISTRICT, CORPS OF ENGINEERS

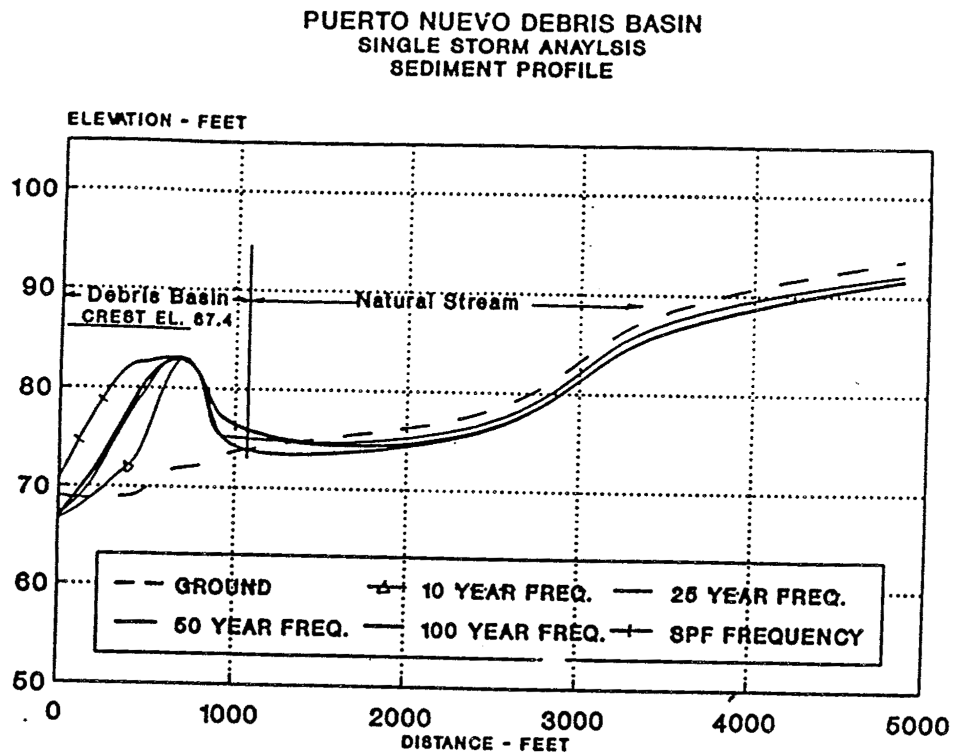


Figure F-12

FIGURE E - 23

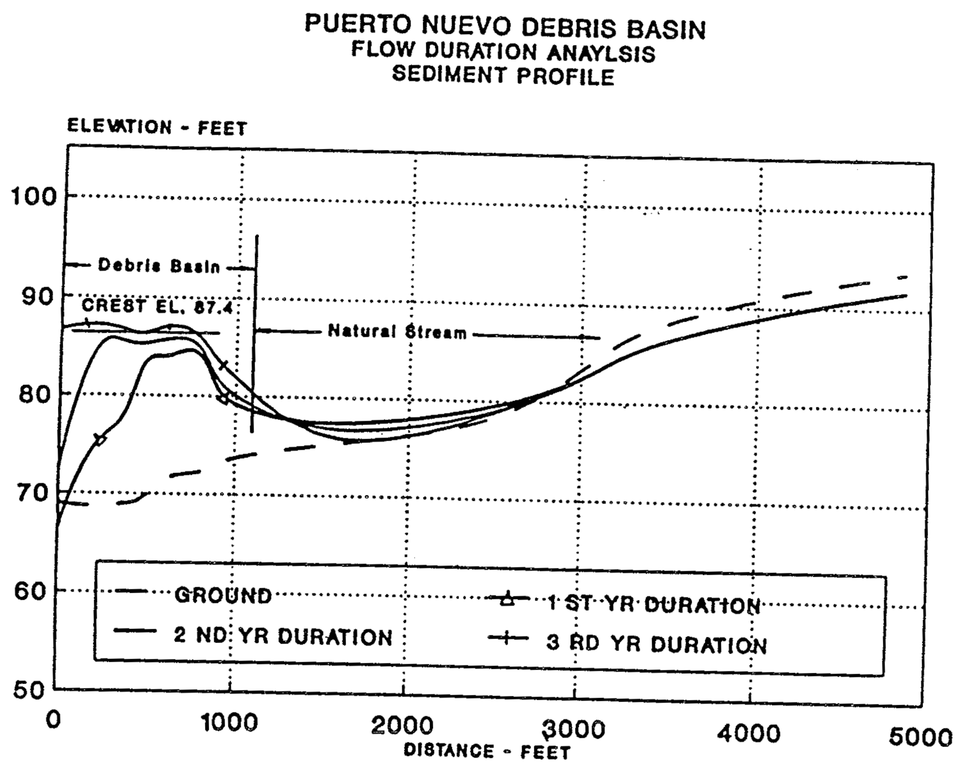


Figure 13

FIGURE E - 24

Steep Stream Riprap Design

Stephen T. Maynord[1]

Introduction and Objectives

Riprap design in steep streams requires consideration of factors such as flow impingement, downslope gravity forces, flow resistance on steep slopes, and alternate methods of estimating velocity that are not required for riprap design in a lower slope environment. For this paper, steep stream riprap design will be divided into the following three categories:

a. Single channels, nonimpinging flow, slopes less than 2 percent or 100 ft/mile.

b. Braided channels, impinging flow, slopes less than 2 percent or 100 ft/mile.

c. Single channels or overflow embankments, nonimpinging flow, slopes between 2 and 20 percent.

Riprap for category *a* streams can be designed using US Army Corps of Engineers guidance for riprap in flood control channels found in Engineer Manual (EM) 1110-2-1601 (Headquarters, US Army Corps of Engineers (HQUSACE), 1991). This guidance departs from the traditional guidance based on shear stress or tractive force and uses a procedure based on local depth-averaged velocity. While the new method can be derived from a modification of the shear stress equations, shear stress is not used explicitly in the new procedure. Local depth-averaged velocity was adopted primarily because local shear stress is difficult to visualize, compute, or measure. From EM 1110-2-1601 the equation for determining stone size is

[1]Research Hydraulic Engineer, Hydraulics Laboratory, US Army Engineer Waterways Experiment Station, 3909 Halls Ferry Road, Vicksburg, MS 39180-6199, (601) 634-3284.

$$D_{30} = S_f C_S C_v C_T d \left[\left(\frac{\gamma_w}{\gamma_s - \gamma_w} \right)^{1/2} \frac{V}{\sqrt{K_1 g d}} \right]^{2.5} \quad (1)$$

where

D_{30} = riprap size of which 30 percent is finer by weight
S_f = safety factor, minimum = 1.1
C_s = stability coefficient for incipient failure, thickness = $1D_{100}$(max) or $1.5D_{50}$(max), whichever is greater, D_{85}/D_{15} = 1.7 to 5.2
= 0.30 for angular rock
= 0.375 for rounded rock (EM 1110-2-1601 incorrect, gives 0.36)
D_{85}/D_{15} = gradation uniformity coefficient
C_v = vertical velocity distribution coefficient
= 1.0 for straight channels, inside of bends
= 1.283 - 0.2 log (R/W) for outside of bends (1 for $R/W > 26$)
= 1.25 downstream of concrete channels
= 1.25 at end of dikes
R = center-line radius of bend
W = water-surface width at upstream end of bend
C_T = blanket thickness coefficient
d = local depth of flow
γ_w = unit weight of water
γ_s = unit weight of stone
V = local depth-averaged velocity
K_1 = side slope correction factor
g = gravitational constant

Riprap designs for categories *b* and *c* require modification of the method presented in EM 1110-2-1601. The objective of this paper is to present riprap design for category *b* and *c* streams.

Riprap Design for Category *b* Streams

For braided streams having impinged flow, the EM 1110-2-1601 procedures require modification in two areas: the method of velocity estimation and the velocity distribution coefficient C_v. All other factors and coefficients presented in the EM are applicable.

The major challenge in riprap design for braided streams is estimating the imposed force at the impingement point. In the EM 1110-2-1601 method, the characteristic imposed force for side slope riprap is the depth-averaged velocity at 20 percent of the slope length up from the toe V_{20}. Although unproven, the most severe bank attack in braided streams is thought to occur when the water surface is at or slightly above the tops of the midchannel bars. At this stage, flow is confined to the multiple channels that often flow into or "impinge" against bank lines or levees. At lesser flows, the depths and velocities in the multiple channels are decreased. At higher flows, the channel area increases drastically and streamlines are in a more downstream rather than into bank lines or levees. Therefore, a method was needed that allows estimation of the average channel velocity when the flow produces a stage at or just above the tops of the midchannel bars. This average channel velocity will be multiplied by an empirical factor to obtain V_{20}, just as in Plate B-33 in EM 1110-2-1601.

The first item that is needed in this method is the discharge that produces a stage near the tops of the midchannel bars Q_{tmcb}. Q_{tmcb} is probably highly correlated with the channel forming discharge concept. In the case of the Snake River near Jackson, Wyoming, Q_{tmcb} is 15,000-18,000 cfs, which has an average recurrence interval of about 2-5 years.

The second item that is needed in this method is cross-section information at sites where the flow is concentrated into one channel against the bank line or levee. In the case of the Snake River, several locations could be found where cross sections had been measured and where the flow was concentrated into a single channel. Using cross-section data to determine the channel area below the tops of the midchannel bars and Q_{tmcb} allows determination of the average channel velocity at the top of the midchannel bars V_{tmcb}.

Field measurements at impingement sites were taken in 1991 on the Snake River near Jackson, Wyoming, and reported in Maynord (in preparation). Flow during these measurements ranged from 14,000 to 16,000 cfs, which produced a stage just below the tops of the midchannel bars. Velocities were measured with electromagnetic velocity meters suspended by a crane that could extend 40 ft from the bank line. Cross sections were not obtained during the 1991 field trip. At eight cross sections measured in 1988, the average channel area below the tops of the midchannel bars was about 2,000 sq ft. Using a Q_{tmcb} of 15,000 cfs resulted in a V_{tmcb} of 7.5 fps. The velocity measurements in 1991 resulted in V_{20} ranging up to 12 fps. The ratio $V_{20}/V_{tmcb} = 12/7.5 = 1.6$, which is almost identical to the ratio shown in Plate B-33 for sharp bendways having $R/W = 2$ in natural channels, and this ratio is recommended for determining V_{20} for impinged flow.

Water-surface measurements on the Snake River at the impingement sites showed that the maximum local water-surface slopes measured over a 100-ft distance averaged 45 ft/mile and ranged from 19 to 82 ft/mile. The downvalley slope of the Snake River in this reach is 19-21 ft/mile.

As stated earlier, two areas of EM 1110-2-1601 require modification for use in impinged flow in braided streams. The second is the velocity distribution coefficient C_v, which varies with R/W in bendways as shown in Plate B-40 in EM 1110-2-1601. In straight laboratory channels having 1V:2H side slopes and channel bottoms with the same riprap size, failure almost always occurred on the channel bottom in stability tests. In laboratory bendways of the Riprap Test Facility, US Army Engineer Waterways Experiment Station, having 1V:2H side slopes, failure generally occurred about halfway up the side slope. Preliminary results from ongoing studies of impinged flow having 1V:2H side slopes showed that failures were initiated higher up the side slope than in the bendway. This suggests that impinged flow has high velocities well up on the side slope, and the 1991 field study (Maynord, in preparation) confirms this observation. The laboratory study of impinged flow is trying to determine the appropriate value of C_v for impinged flow. Until that time a value of C_v of 1.25, which is close to a bendway having $R/W = 2$, is recommended.

For the Snake River near Jackson, Wyoming, the required riprap size using the procedures presented herein is as follows:

Input: $V_{20} = 1.6(15{,}000/2000) = 12$ fps, depth at $V_{20} = 10$ ft, specific weight = 155 pcf, $C_v = 1.25$, $C_s = 0.30$, $C_t = 1.0$, $S_f = 1.1$, 1V:2H side slope, thickness = $1D_{100}$, use ETL 1110-2-120 gradations given in Table 3-1 of EM 1110-2-1601.

Result: Required $D_{30} = 1.09$ ft, ETL $D_{30}(\min) = 1.10$ ft, thickness = 27 in., $W_{50}(\min) = 185$ lb.

This compares with the existing riprap that has an average size of less than 100 lb according to US Army Engineer District, Walla Walla (1987). New riprap placement along the Snake River generally uses riprap having $W_{50} = 400$ lb with thickness of 42 in. at the toe and 24 in. at the top.

Riprap Design for Category *c* Streams

For single channels or overflow embankments, slopes greater than 2 percent are outside the range of direct applicability of EM 1110-2-1601 because of the importance of the downslope gravity component and the effect of steep slopes on flow resistance. Overflow embankment riprap stability tests have generally been limited to a maximum slope of 20 percent. The most recent tests were conducted by Abt et al. (1986) and Abt et al. (1988). While a 20 percent slope may seem large for loose riprap, highway engineers have questioned this author about design guidance for riprap placed on slopes approaching 40 percent. Using Abt's data and dimensional analysis results in the following empirical equation

$$D_{50} = \frac{2.26 S^{0.555} q^{2/3}}{g^{1/3}} \qquad (2)$$

or in terms of D_{30} used in EM 1110-2-1601

$$D_{30} = \frac{1.95 S^{0.555} q^{2/3}}{g^{1/3}} \tag{3}$$

where S is the slope of the bed and q is the unit discharge. Both equations can be used in any consistent set of units, both fall on the conservative side of the data, and both are restricted to a thickness of $1.5D_{100}$, angular rock, specific weight of 167 pcf, 6-in. gravel filter beneath riprap, D_{85}/D_{15} from 1.7 to 2.7, slopes from 2 to 20 percent, uniform flow on a downslope with no tailwater, and average riprap size less than 6 in. The comparison of Equation 3 with the data is shown in Figure 1. One of the problems with this approach is that different specific rock weight, blanket thickness, and gradation uniformity cannot be used with this approach.

An alternative approach would be to use the EM 1110-2-1601 procedure to address other specific weights, thickness, and gradation but to include the appropriate factors for downslope gravity effects and a resistance equation for flow on steep slopes. From Ulrich (1987), the appropriate K_1 factor to use in Equation 1 is

$$K_1 = \text{Cos}\,\alpha \left[1 - \frac{\gamma_s}{\gamma_s - \gamma_w} \frac{\text{Tan}\,\alpha}{\text{Tan}\,\phi}\right] \tag{4}$$

When α is the angle of the channel bottom from horizontal and ϕ is the angle of repose of the riprap revetment. From Maynord (1988) the appropriate ϕ for riprap revetments is about 53 degrees. Using Abt et al. (1986), flow resistance data and dimensional analysis result in the following modification of the Strickler equation

$$n = 0.07(D_{50}S)^{1/6} \tag{5}$$

which is applicable to slopes between 2 and 20 percent. Combining Equations 1, 4, and 5, using $q = Vd$, and $D_{50} = 1.2D_{30}$ results in

$$D_{30} = C' \frac{q^{2/3} S^{0.432}}{g^{1/3} K_1} \tag{6}$$

where

$$C' = 5.3(S_f C_v C_t C_s)^{0.785} \left[\frac{\gamma_w}{\gamma_s - \gamma_w}\right] \tag{7}$$

Note the similarity of Equation 6 and Equations 2 and 3 and that Equation 6 was derived without using Abt's et al. stability data. Also note that the slope effect in Equation 6 is also part of the K_1 factor in the denominator. Equation 6 is limited to slopes from 2 to 20 percent, angular rock, 6-in. gravel filter beneath riprap, uniform flow on downslope with no tailwater, and average riprap size less than or equal to 6 in. The comparison of Equation 6 using $S_f = 1.1$ (minimum safety factor), $C_v = 1$, $C_T = 0.84$ (for thickness $= 1.5D_{100}$), $C_s = 0.30$ (for angular rock), specific stone weight= 167 pcf, and $\phi = 53$ degrees with Abt's et al. data is shown in Figure 1 as the modified EM 1110-2-1601 curve. Equation 6 fits the observed data as well as the empirical approach given by Equation 2 or 3 and allows variation of stone size with unit weight, blanket thickness, etc.

Abt et al. (1988) presents a flow concentration factor that varies from 1 to 3 that is multiplied by the unit discharge when the inflow is not uniform across the approach channel. Although guidance is lacking on the amount of flow concentration for a given geometry, some degree of flow concentration should be expected. Riprap on steep slopes should be relatively uniform with $D_{85}/D_{15} \leq 2.5$. Additional studies are needed to extend Equations 2, 3, or 6 to larger riprap sizes.

Consider a 10-ft-wide downslope having a 10 percent slope and a total discharge of 25 cfs. Rock protection will be placed to a thickness of $1.5D_{100}$, and have a unit weight of 165 pcf. Using a flow concentration factor of 1.25 results in a unit discharge of $1.25(25/10) = 3.13$ cfs/ft. Using Equation 3, the required $D_{30} = 0.37$ ft. Using the modified EM 1110-2-1601 procedure given by Equation 6, the required $D_{30} = 0.34$ ft. In either case, a typical gradation having $D_{30}(\min) \geq 0.34$ ft would have $D_{100}(\max)$ of about 9 in. and a blanket thickness of $1.5(9) = 13$-14 in.

<u>Summary and Conclusions</u>

Riprap design for single channels, nonimpinging flow, and slopes less than 2 percent should use guidance presented in EM 1110-2-1601.

Riprap design for braided channels, impinged flow, and slopes less than 2 percent should use the velocity estimation method presented herein and $C_v = 1.25$ in the EM 1110-2-1601 procedure.

Riprap design for single channels or overflow embankments, nonimpinged flow, slopes between 2 and 20 percent, uniform flow on a downslope with no tailwater, and average rock size less than or equal to 6 in. should use either the empirical method in Equation 2 or 3 or the modification of the EM 1110-2-1601 method given in Equation 6.

Conversion Factors from U.S. Customary to SI Units

To convert (1)	To (2)	Multiply by (3)
Cubic foot per second (cfs)	Cubic meter per second (m^3/s)	0.03
Degree	Radian	0.02
Foot (ft)	Meter (m)	0.31
Inch (in.)	Meter (m)	0.03
Pound (mass) (lb)	Kilogram (kg)	0.45
Pound (mass) per cubic foot (pcf)	Kilogram per cubic meter (kg/m^3)	16.02
Square foot (sq ft)	Square meter (m^2)	0.09

References

Abt, S. R., Ruff, J. F., Khattak, M. S., Wittler, R. J., Shaikh, A., and Nelson, J. D. 1986. "Environmental assessment of uranium recovery activities—riprap testing, Phase I," Report No. CER85-86SRA-JFR-MSK-RJW-AS-JDN24, Department of Civil Engineering, Colorado State University, Fort Collins, CO.

Abt, S. R., Wittler, R. J., Ruff, J. F., LaGrone, D. L., Khattak, M. S., Nelson, J. D., Hinkle, N. E., and Lee, D. W. 1988. "Development of riprap design criteria by riprap testing in flumes: Phase II," NUREG/CR-4651, ORNL/TM-10100/V2, Vol 2, Prepared for US Nuclear Regulatory Commission by Colorado State University, Fort Collins, CO, and Oak Ridge National Laboratory, Oak Ridge, TN.

Headquarters, US Army Corps of Engineers. 1991. "Hydraulic Design of Flood Control Channels," EM 1110-2-1601, US Government Printing Office, Washington, DC.

Maynord, S. T. 1988. "Stable riprap size for open channel flows," Technical Report HL-88-4, US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Maynord, S. T. "Flow impingement, Snake River, Wyoming" (in preparation), US Army Engineer Waterways Experiment Station, Vicksburg, MS.

Ulrich, T. 1987. "Stability of rock protection on slopes," *Journal of Hydraulic Engineering, ASCE*, 113(HY7), 879-891.

US Army Engineer District, Walla Walla. 1987. "Jackson Hole, Wyoming, Flood Protection Project," Briefing book provided to Committee on Channel Stabilization held in July, 1987 in Jackson, Wyoming.

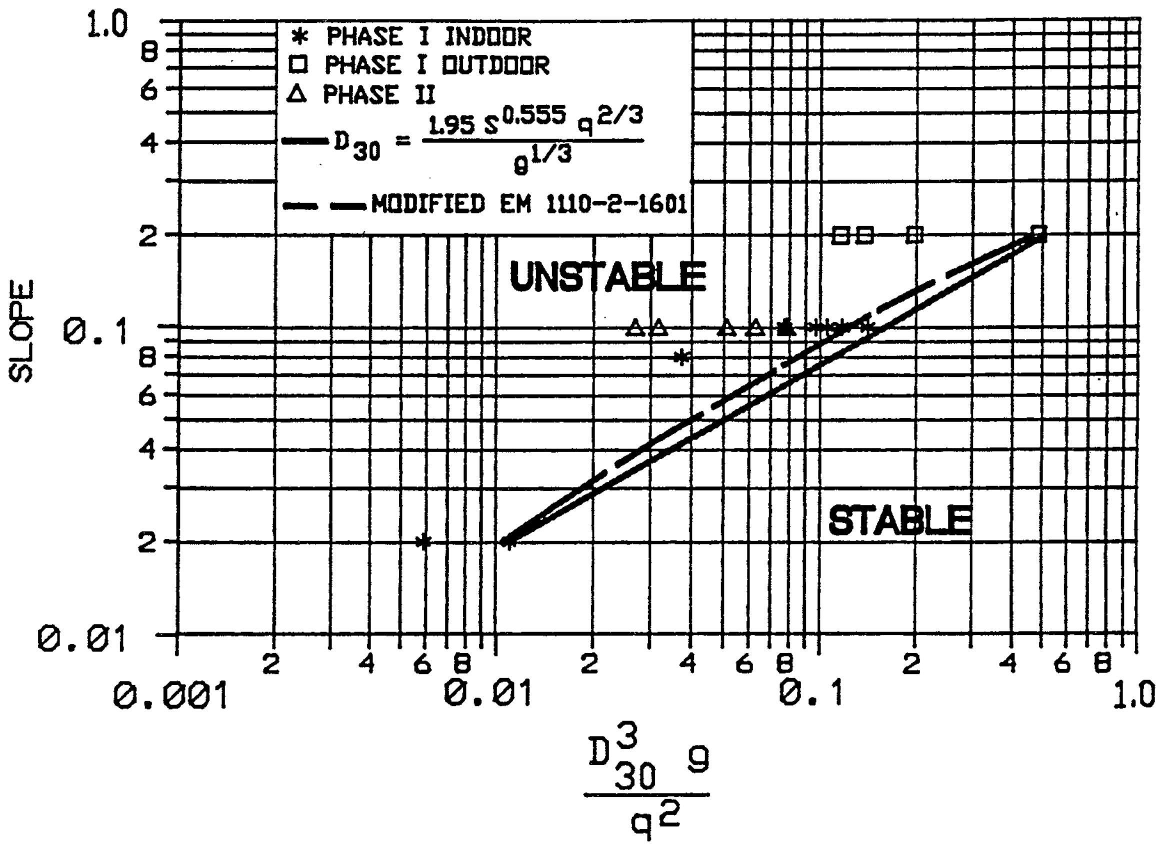


FIGURE 1. RIPRAP STABILITY FOR DOWNSLOPES FROM 2 TO 20 PERCENT

FLAMINGO-TROPICANA ALLUVIAL FAN PROJECT

by

Brian Tracy[1]

INTRODUCTION

Purpose. This paper presents the with- and without-project feasibility designs for the Flamingo-Tropicana Flood Control Project. The proposed Flamingo-Tropicana Project consists of a system of debris and detention basins on several alluvial fans inter-connected by high velocity concrete channels. The project site is located just west of the downtown portion of the City of Las Vegas, Nevada. A significant part of the project benefits come from "Flood Proofing Costs Reduced". In this case this benefit is based upon how the Federal Emergency Management Agency (FEMA) enforces its Alluvial Fan Zone Regulations. Assessment of without-project conditions required the design of "decentralized" flood protection features on the alluvial fans. The with-project condition required the design of a centralized system. The Flood Proofing Costs Reduced benefit consists of the difference in cost between the less efficient without-project system and the more efficient with-project system.

Key Issues. Key design issues involved 1) evaluating existing flooding conditions on several adjacent alluvial fans, 2) producing without-project designs for dealing with the existing flooding conditions while meeting regulatory requirements and 3) developing an efficient with-project flood control design. The with-project design must deal with the sediment and runoff environment on the alluvial fans while also minimizing the adverse impacts the project would create in greater than design events.

PHYSICAL SETTING

The Flamingo-Tropicana Washes watershed is located on the western side of the Las Vegas Valley in Clark County, Nevada, west of the city of Las Vegas. It is bounded on the north by the La Madre Mountain range, on the west by the Spring Mountains, and on the south by a lower divide formed by alluvial materials eroded from the Spring Mountains to the west. The mountains on the northern and western boundaries of the watershed are rugged and steep. Alluvial fans form the transition from the steep mountains to the urbanized areas of the gently sloping valley floor to the east. The Blue Diamond Mountains are an unusual feature located within the watershed near the western boundary. They constitute an interior area of high ground which causes some

---

[1]Chief, Hydraulics Section, Los Angeles District, U.S. Army Corps of Engineers

runoff to drain west, then north to Redrock Wash or south to Blue Diamond Wash before it can join the general direction of flow to the east. Figure 1 shows the Flamingo-Tropicana Washes watershed.

Flamingo Wash begins as an ephemeral desert stream originating in the Blue Diamond Mountains and terminating approximately 17 miles downstream at its confluence with Las Vegas Wash. It flows generally east down the alluvial fan area and into urban Las Vegas where it eventually terminates at its confluence with Las Vegas Wash. It is interesting to note that at one point Flamingo Wash flows under and through the parking garages of some casinos on Las Vegas Boulevard ("The Strip"). Urban runoff and leakage from the groundwater aquifer make the stream perennial in its lower reaches. Tropicana and Red Rock Washes are its principal tributaries.

Red Rock Wash drains the area to the north of the Blue Diamond Mountains and joins Flamingo Wash near the base of the alluvial fan. Tropicana Wash begins as a distributary of Blue Diamond Wash. It flows generally northeast down the alluvial fan area, enters urban Las Vegas and eventually flows into Flamingo Wash.

The alluvial fan area on the eastern side of the Blue Diamond Mountains may be somewhat atypical due to the widespread presence of outcrops and subsurface layers of caliche. Caliche is a naturally cemented material which varies widely in its durability. It is believed that the caliche has a pronounced influence on the behavior of the alluvial fans. It is very likely that it changes the patterns of deposition, avulsions and channel formation that would occur on an unconstrained alluvial fan, but this effect is almost impossible to quantify.

In the urban areas downstream of the alluvial fans the Flamingo and Tropicana Washes are locally disturbed in so many places that it is difficult and perhaps pointless to develop any sense of the general morphology of these streams. During flood events, these streams have shown a tendency to both locally erode and locally deposit.

WITHOUT-PROJECT CONDITIONS

Overflow Analysis. A traditional HEC-2 overflow analysis was conducted for the well defined reaches of the Flamingo and Tropicana Washes where they flow through the urbanized areas below the alluvial fan area. The 25-, 50-, 100- and 500-year frequency overflows were delineated for the purpose of evaluating inundation reduction benefits. Non-damaging discharges were also determined and plotted on profiles of the streams for use in the economic evaluation and also as a plan formulation tool.

Because inundation reduction benefits for the alluvial fan area were expected to be minimal, without-project multiple frequency overflows were not developed for the alluvial fan part of the study reach. Only the 500-year frequency event was delineated for purposes of evaluating the impact of the proposed project upon greater than design events. The 500-year overflow was estimated by developing separate HEC-2 models for each hydrologic subarea on the alluvial fans. The cross sections taken for geometric input into the model were extended all the way across each subarea. The local 500-year discharge was used for each HEC-2 model. The resulting 500-year overflow covers the entire alluvial fan area. This overflow does not represent a single event, but a composite of all the local 500-year floods on the alluvial fans.

<u>FEMA Regulations</u>. An economic analysis determined that the alluvial fans in the upper part of the study reach were going to develop entirely by the year 2026 in spite of the lack of a centralized flood control system and the regulatory requirements that FEMA imposes in alluvial fan zones. FEMA enforces its alluvial fan regulations by requiring a developer to provide protection from flooding for the computed probability 100-year event emanating from the entire drainage area tributary to the proposed development. The developer must do this without diverting or discharging concentrated flows onto any adjacent property in order to obtain a letter of map revision from FEMA.

<u>Without-Project Designs</u>. Because the alluvial fan area will develop regardless of the presence or absence of a centralized flood control project, the project can claim "Flood Proofing Costs Reduced" benefits for the difference between the total costs of what the developers would build piecemeal to meet FEMA requirements to protect their individual properties and the cost of an efficient central flood control system. In order to estimate this cost, it was necessary to produce "without-project designs" of what the developers would do.

Property ownership on the alluvial fans consists of a patchwork of various size parcels. It was assumed that each developer would design and build a flood protection system optimized to a his or her individual parcel size. Because flows may not be diverted onto adjacent properties, they must be collected at the upstream side of a given property and conveyed through the parcel. It was assumed that street conveyance would not be acceptable. Because FEMA requirements do not allow the developer to discharge concentrated flows onto his neighbors, the design must "re-disburse" the flood flows collected at the upstream side of the property before releasing them.

The resulting general design is sometimes referred to as the "moat plan". It consists of drop inlet collector channels at the upstream side of the parcel, conveyance channels around and through the parcel, and dispersion channels at the downstream side of the parcel. The dispersion channels discharge the flows on the downstream side of the parcel in a distribution pattern similar to the one in which they entered at the upstream side. A series of designs for various standard parcel sizes was developed for each drainage subarea on the fan. The design discharge for each design was based upon the local computed probability 100-year (70-year expected probability) discharge divided by the average subarea width and multiplied by the length normal to the flow of the parcel in question.

The cost of each design was estimated, and based upon the current actual distribution of parcel sizes in each drainage subarea the total cost of the decentralized without-project system was determined. The costs of debris control in the without project designs were ignored. This is a conservative assumption from the point of view of project economics.

FEASIBILITY DESIGN

System Overview. The proposed project consists of a system of four inter-connected detention basins (Red Rock, Blue Diamond, Flamingo and Tropicana), four debris basins, and eight high velocity concrete channels which provide a 100-year computed probability level of flood protection on the alluvial fans draining to the Flamingo and Tropicana Washes. The system will work in conjunction with a locally designed "secondary" system of lateral collector channels which will meet specified project performance criteria. Except for some mitigation features, the structural features of the proposed project are all located on the alluvial fans west of most of the current intense urban development. While these features will greatly alleviate the current urban flooding, they do not eliminate it. There are not sufficient incremental benefits for inundation reduction to justify the necessary improvements through most of the current urban area draining to Las Vegas Wash through the Flamingo and Tropicana Washes. The proposed project, however, will make it possible for the local sponsor to manage this problem independently. Refer to Figure 2 for a schematic of the proposed system.

Hydrology. Each element of the project design is sized to handle the local, computed probability 100-year discharge. The flood control system as a whole has been sized to evenly distribute the "stress" on the system during a regional 100-year computed probability event. Specifically, the storage and outflow characteristics of the various detention basins were adjusted so that each basin would have a similar percentage of storage utilized throughout the design event.

Collection. Runoff is collected into the system in one of three ways: 1)Directly into the detention/debris basins, 2) directly into the high velocity concrete main channels and 3) indirectly into the main channels through the secondary lateral collector system.

The first two collection methods are fairly straightforward, however the secondary lateral collection system is rather unusual. In order for the project to provide the benefits it claims, there must not be any location related development constraints. In other words, once the system is completed, development can occur anywhere on the fan without the need to consider what other development has taken place. In order for this to be true, all of the lateral collector channels must be in place initially. However, because the fan will not be fully developed until 25 years after the project is completed, there are not enough benefits to support the initial construction of the entire final lateral system. Therefore, a temporary "initial" lateral system will be constructed.

The initial lateral system will consist of simple earthen trenches. Its purpose will be to divide the drainage area of the fan so that a developer will only need to deal with the runoff from his drainage subarea. These initial laterals are not expected to be stable. During the 25 year transition period, some of these channels will experience significant runoff events. They will probably try to meander by bank erosion and the rivulets draining to them will probably headcut during such events. Because the adjacent property will be undeveloped however, this will not affect their contribution to the project benefits. When the property adjacent to a given initial lateral is developed, at that time the final lateral will be installed. This will be a permanent, stable collector of some kind. The major concerns with the initial lateral channels are 1) whether they will tend to plug with sediment during a design event and 2) the amount of sediment that they will deliver to the main system.

The "Stable Channels Module" of the SAM program was used to evaluate the stability of the initial lateral channels. The results indicate that the response of the channels will be different depending upon the magnitude of the flood event. Smaller events will tend to deposit sediment in the channels but larger events such as the design event will tend to degrade the channel inverts and will not cause significant blockage. Therefore, the conclusion reached was that the lateral system would function adequately but that some operation and maintenance expense would probably be required until the final lateral system is fully implemented.

The volume of sediment that will be delivered by the initial lateral system was addressed by performing a sediment budget analysis for the design flood event. Sheet and rill erosion on the alluvial fans were estimated using the Universal Soil Loss and Modified Universal Soil Loss equations. Gully erosion was estimated by using a method developed by the Soil Conservation Service and published in ASCE Manual 54. Bed degradation within the laterals was estimated using HEC-6. Bank erosion in the initial laterals was not independently estimated. It was found that the sediment supply to the initial laterals from all other sources was generally less than the transport capacity of the laterals, consequently it was assumed that the difference would be made up by bank erosion. Effectively, the estimate of the material delivered by the laterals is therefore equal to their sediment transport capacity.

The initial and final secondary lateral collector channels will be designed and constructed by the local sponsor or under their direction. The Corps will provide specific performance criteria but will not otherwise constrain the design. This will make it possible for the final lateral channels to be integrated into the final development in a manner that is most acceptable to the local interests. The laterals could easily be designed to serve additional purposes such as parks, walking trails, golf courses, etc...

Debris Management. One of the most important considerations in designing a flood control system on an alluvial fan is the management of debris at the upstream project inlets. A centralized flood control system must halt the natural process of sediment deposition over a widespread fan shaped area. In an urban setting this natural process is extremely destructive. In addition, this particular system must also deal with sediment which is generated on the fan itself during the initial 25 year period before the fan becomes completely developed.

The design which is proposed for the Flamingo-Tropicana Washes is a debris basin-high velocity concrete channel system. This type of system is used extensively throughout the Los Angeles District, particularly in the Los Angeles metropolitan area. It is a well tested, highly reliable concept.

The most critical part of this concept is the debris basin. The debris basin must be designed and sized to induce sediment to deposit in such a way that it halts the natural process of channel avulsion at the apex of the alluvial fan for events up to the design event. In the Los Angeles District, debris basins are usually designed and sized empirically. The empirical relations are based upon experience with over eighty existing debris and detention basins, mostly located in the Los Angeles area. Using basin clean-out records, regression analyses have been performed which provide relationships between debris yield and a number of pertinent parameters for specific flood events. This procedure was used to determine the 100-year computed probability debris yield for the debris basins and upstream detention basins proposed for the Flamingo-Tropicana Washes project.

Another important empirical observation is that generally the sediment deposits which have filled the existing debris basins have slopes which are approximately one half of the natural stream slope. Common sense dictates that once a debris basin becomes filled, the slope of the sediment in the debris basin will approach that of the natural stream bed asymptotically as time passes. But for the short duration events experienced in the Los Angeles District the slopes rarely exceed the one half of the natural slope rule. In combination with the storage-elevation characteristics of each debris basin site and the estimated debris production, this rule was used to determine the required debris basin embankment heights or pit depths.

Where site conditions permit, it is generally preferable to limit the debris basin embankment heights to under six feet. This can be done by excavating rather than raising the embankment to provide storage. By keeping the embankment low, the consequences associated with embankment failure can be minimized and the construction of an emergency spillway can be avoided. This design approach was possible with two of the four debris basins in the proposed Flamingo-Tropicana system.

The second part of the system is the high velocity concrete channel. High velocity channels were selected for this system for reasons outlined below under the conveyance heading of this paper. From the point of view of debris management however, they are highly desirable for two reasons. First, they eliminate the channels themselves as a source of debris because they preclude channel erosion and the growth of vegetation. Second, because of their high sediment transport capacities, they can easily transport sediment which passes through properly sized debris basins or which enters through the initial lateral collector system.

Detention. Detention can be a highly cost effective measure in an alluvial fan environment if adequate sites are available. The reason for this is that the flood hydrographs generally have high peak discharges with relatively little volume. Providing storage can therefore drastically reduce peak discharges. Unfortunately, because high intensity rainfall can occur very locally, (i.e. just downstream of the detention basin), the effects of this method of reduction are very local. In the case of this project, four effective detention sites were identified. Two are upstream sites, one is centrally located, and one is at the downstream end of the major structural improvements.

Conveyance. Conveyance between the detention basins and the point of disposal for the lateral collectors is provided by a network of high velocity concrete channels. This part of the system is probably the most controversial. Other types of channels are often preferable for aesthetic, environmental or the desire for multiple use reasons. From a purely flood control point of view, however, this type of channel is usually the most cost effective and reliable on an alluvial fan.

The main alternative to a high velocity concrete channel is a soft bottom channel. Because of steep slopes on alluvial fans, a soft bottom channel would have to be relatively wide and would require many drop structures. Construction costs can be just as high as for concrete channels and land costs are usually much higher. Wide, soft bottom channels are less reliable because of the potential for a meandering low flow channel to develop which can attack the channel boundaries. The side slope revetment would require deep toe downs to protect against this. In addition, bridges over this type of channel would require relatively long spans with many piers in the flow (the alternative is a dip crossing which is very unsafe in an urban area). Soft bottom channels require regular clearing which can be costly and is often controversial in itself. Vegetation in the channels can also cause trash problems at bridge piers and other structures in the flow. Because they have moveable boundaries and relatively low sediment transport capacities, soft bottom channels can develop local deposition and scour problems at confluences, side drain inlets, bends and bridges which makes them less reliable. For these reasons the high velocity concrete channels were selected as the basis for establishing the limit of federal interest in the project.

One of the high velocity channels is actually a diversion channel. This channel diverts the outflow from the Flamingo Detention Basin from Flamingo Wash to the Tropicana Detention Basin which discharges into Tropicana Wash. This alignment modification accomplishes two things: 1) it bypasses flood flows around the constricted reach of Flamingo Wash thus achieving downstream inundation reduction benefits and 2) it isolates the impact of long duration detention basin releases to Tropicana Wash and only the downstream part of Flamingo Wash.

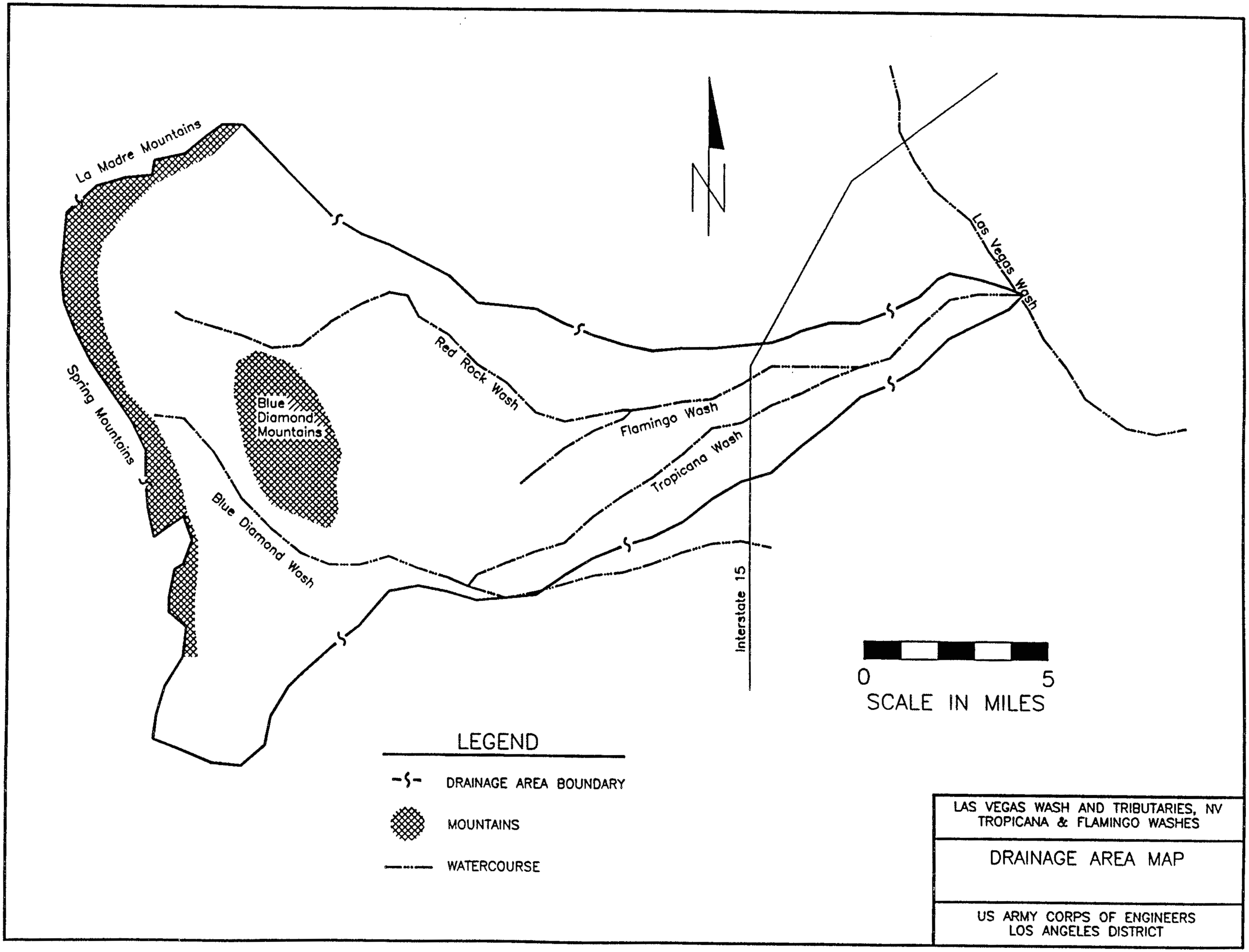
<u>Impact Mitigation</u>. The principal adverse impact of the project is that low peak, long duration (eight days for the design event) clear water discharges will be released from the Tropicana Detention Basin into Tropicana Wash. This will result in a pattern of aggradation and degradation in the downstream channel that is generally more severe than under pre-project conditions. To mitigate for this condition a combination of grade control and bank protection measures will be installed in Tropicana and Flamingo Washes downstream of Tropicana Basin.

<u>Greater-than-design Events</u>. Certain features have been incorporated into the project design to accommodate greater than design events. The secondary lateral collection system will be designed to preclude greater than design events from entering the system. Open channels will be designed without freeboard and pressure flow side drains will not be oversized. The main channels are aligned parallel to the natural pre-project flow paths to the extent possible to avoid intercepting runoff from greater than design events. This is not always possible, however, so the design will include reaches with overtopping zones to allow flows which exceed the channel design capacity to escape the system. These zones will generally be located where the channels cross the natural flow paths. All of the detention basins and debris basins with embankments will have spillways

capable of passing the probable maximum flood.

CONCLUSION

The proposed project consisting of a series of debris and detention basins interconnected by high velocity concrete channels is efficient and cost effective. It provides a point of disposal for a 100-year computed probability flood control system on the alluvial fans in Clark County west of the city of Las Vegas, Nevada. It also significantly reduces the flood problem along Flamingo and Tropicana Washes within Las Vegas. The design considers debris and sediment movement, greater than design flood conditions and the downstream adverse impacts due to long duration low peak discharge releases.



La Madre Mountains
Spring Mountains
Blue Diamond Mountains
Red Rock Wash
Flamingo Wash
Tropicana Wash
Blue Diamond Wash
Las Vegas Wash
Interstate 15
N
0
5
SCALE IN MILES
LEGEND
DRAINAGE AREA BOUNDARY
MOUNTAINS
WATERCOURSE
LAS VEGAS WASH AND TRIBUTARIES, NV
TROPICANA & FLAMINGO WASHES
DRAINAGE AREA MAP
US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE 1

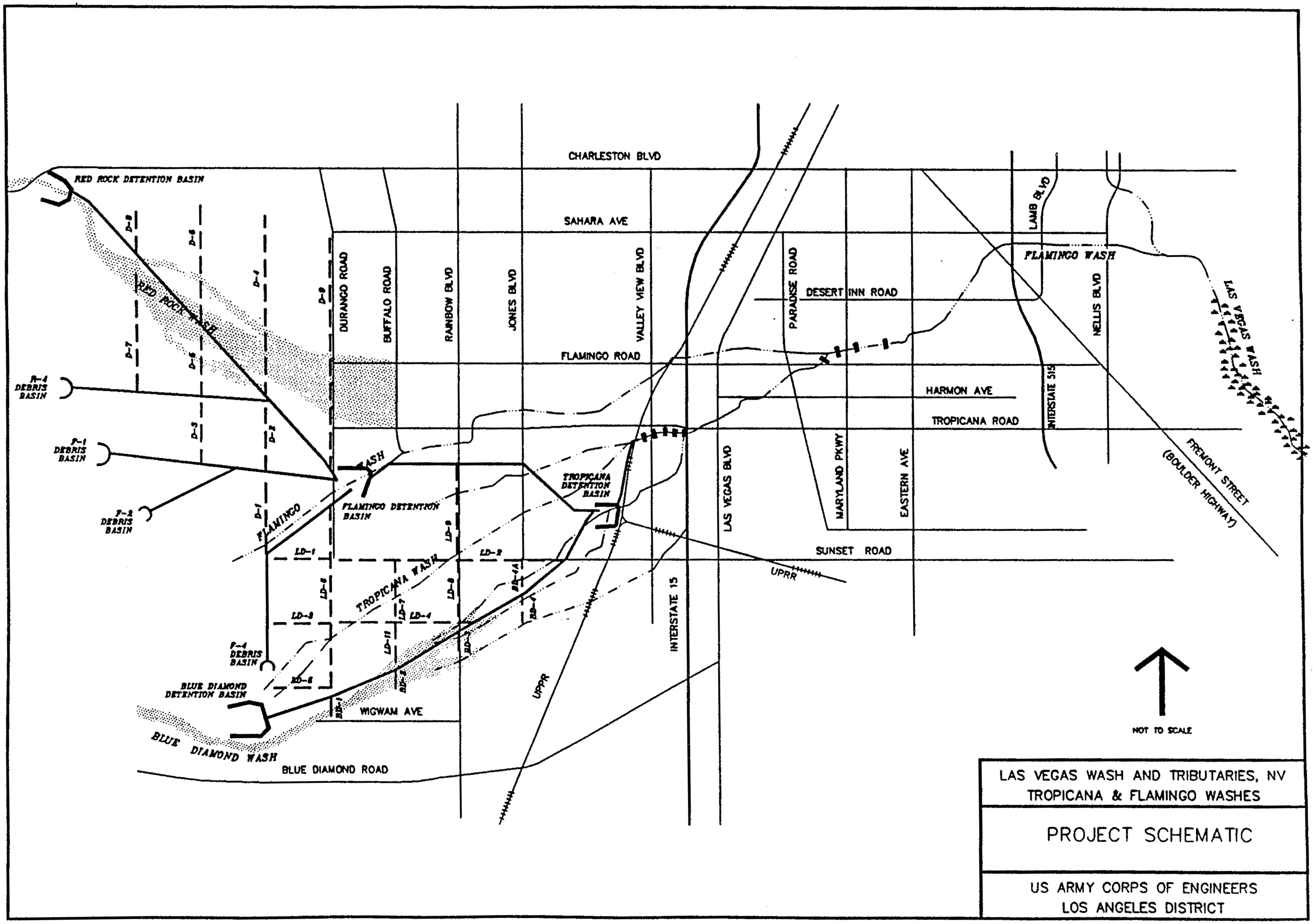

CHARLESTON BLVD
SAHARA AVE
DESERT INN ROAD
FLAMINGO ROAD
HARMON AVE
TROPICANA ROAD
SUNSET ROAD
WIGWAM AVE
BLUE DIAMOND ROAD
DURANGO ROAD
BUFFALO ROAD
RAINBOW BLVD
JONES BLVD
VALLEY VIEW BLVD
INTERSTATE 15
LAS VEGAS BLVD
PARADISE ROAD
MARYLAND PKWY
EASTERN AVE
LAMB BLVD
NELLIS BLVD
INTERSTATE 515
FREMONT STREET
(BOULDER HIGHWAY)
UPRR
RED ROCK DETENTION BASIN
RED ROCK WASH
FLAMINGO WASH
TROPICANA WASH
BLUE DIAMOND WASH
LAS VEGAS WASH
FLAMINGO DETENTION BASIN
TROPICANA DETENTION BASIN
BLUE DIAMOND DETENTION BASIN
R-4 DEBRIS BASIN
F-1 DEBRIS BASIN
F-2 DEBRIS BASIN
F-4 DEBRIS BASIN
NOT TO SCALE
LAS VEGAS WASH AND TRIBUTARIES, NV
TROPICANA & FLAMINGO WASHES
PROJECT SCHEMATIC
US ARMY CORPS OF ENGINEERS
LOS ANGELES DISTRICT

FIGURE 2

Numerical Simulation of Mudflows from Hypothetical Failures of the Castle Lake Debris Blockage Near Mount St. Helens, WA

Presented By

Gary W. Brunner, M.S., P.E.
U.S. Army Corps of Engineers
Hydrologic Engineering Center
609 Second Street
Davis, California 95616

Foreword and Credits

This report was prepared at the request of the Hydrologic and River Engineering Section of the Portland District, U.S. Army Corps of Engineers. The HEC was asked to assist in the assessment of hypothetical mudflow events that might occur if the debris blockage presently containing Castle Lake near Mount St. Helens, WA were to fail. The Portland District Corps of Engineers, at the request of the U.S. Forest Service (managers of the Mount St. Helens National Volcanic Monument), undertook their initial studies in 1988 to analyze the existing conditions of the blockage, determine the degree of risk for downstream flooding posed by Castle Lake, evaluate alternatives to reduce that risk, and recommend a solution for reducing the risk. The primary purpose of this study was to estimate the potential for flooding downstream from the Corps' Sediment Retention Structure (SRS) for various hypothetical lake breaching scenarios. It is not the intent of this investigation to evaluate any aspect of the risk of failure. It merely quantifies the downstream flood potential for various hypothetical breaching scenarios. Results from this investigation are to be used by USFS and USACE managers to decide what alternatives may be effective in reducing the flooding potential in communities downstream from the SRS.

Information and data used during this investigation and presented in this report were obtained from the Portland District Corps of Engineers, the U.S. Forest Service, the U.S. Geological Survey and the Washington State Department of Ecology.

Robert C. MacArthur , Gary Brunner and Doug Hamilton conducted the investigation and wrote this report. Messrs. MacArthur and Brunner work at the Hydrologic Engineering Center in Davis, CA. Doug Hamilton is a principal engineer with RIVERTECH, Inc. in Laguna Hills, CA and provided technical assistance to the Hydrologic Engineering Center during this investigation. Ron Mason was the Project Manager for the Portland District Corps of Engineers and Mr. John Steward was the Project Manager for the U.S. Forest Service. Vernon Bonner was the Chief of the Training Division during the study and Mr. Darryl Davis was the Director of the Hydrologic Engineering Center during the investigation.

The investigation reported herein was conducted by the Hydrologic Engineering Center in Davis, California at the request of the Portland District, Corps of Engineers and the Gifford Pinchot National Forest, U.S. Forest Service. Funding for this study was provided by the U.S. Forest Service.

Numerical Simulation of Mudflows from Hypothetical Failures the Castle Lake Debris Blockage Near Mount St. Helens, WA

1. Study Purpose

The May 18, 1980 eruption of Mount St. Helens, WA, produced a debris avalanche that flowed down the North Fork Toutle River damming several tributary streams. The blockage at the confluence of South Fork Castle Creek and Castle Creek produced a natural debris dam approximately 190 feet high. Figure 1 shows the general study area near Mount St. Helens and the location of Castle Lake. Snow melt and runoff waters captured behind the blockage quickly formed a lake. To prevent overtopping and a potentially catastrophic failure of the blockage retaining Castle Lake, the U.S. Army Corps of Engineers (USACE) constructed an emergency spillway in October 1981 at the eastern end of the blockage to stabilize the lake at elevation 2,577 feet NGVD. Studies by the U.S. Geological Survey (USGS) indicated that "the blockage is potentially unstable against failure from piping due to heave and internal erosion when groundwater levels are seasonally high" and that an earthquake of 6.8 or greater might initiate such a failure (Laenen and Orzol, 1987). If the Castle Lake blockage were to fail rapidly by the mechanism suggested by the USGS, approximately 18,500 acre-feet (AF) of stored water in the lake could create a mudflow flood event in the North Fork Toutle River. The USGS (Laenen and Orzol, 1987) estimates that an event of this nature could result in a peak discharge of 2,100,000 cfs at the Corps' N-1 debris retention dam ten miles downstream from Castle Lake (see Figure 1) and possibly lead to downstream flooding.

In the wake of the Mount St. Helens eruption, the Corps developed a long-term flood control and navigation maintenance plan. A major component of that plan is the $56.5 million Sediment Retention Structure (SRS) designed to trap the huge amounts of sediment expected to continue to move down the North Fork Toutle River. The SRS was designed to capture runoff-induced sediment from the blast zone, thus preventing sediment deposition in the Cowlitz and Columbia Rivers. Without the SRS, sediment materials could continue to accumulate in the rivers below the SRS thus reducing their flood routing and navigation capacities. There was additional concern that failure of the Castle Lake blockage resulting in the possible occurrence of a mudflow event could jeopardize the safety and performance of the SRS or perhaps lead to flooding in communities downstream from the SRS.

The purpose of this study is to evaluate the hydraulic characteristics of mudflow events resulting from the hypothetical failure of Castle Lake and to examine the ability of the SRS to capture and pass such events through its spillway for various initial conditions at Castle Lake and in the SRS. More specifically, the study is to: (1) determine if flows will exceed the present spillway capacity of the SRS, (2) determine if the SRS will be overtopped during various breaching scenarios, (3) estimate how the peak discharge in communities downstream from the SRS will be affected by the presence of the SRS, (4) evaluate the routing effects on the resulting mudflow hydrographs due to lowering the initial Castle Lake levels at the time of breaching, and (5) evaluate the performance of the SRS during these various events when the SRS is empty of water and sediment (existing conditions), or full of sediment deposits up to the spillway crest.

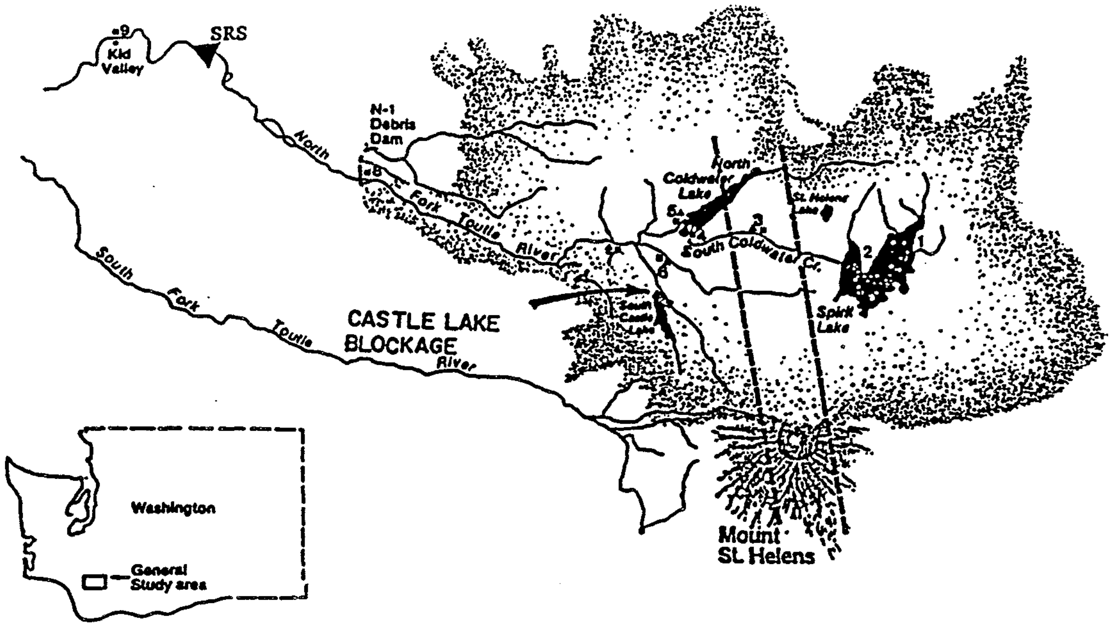


Figure 1
General Study Area

2. Approach

The Hydrologic Engineering Center (HEC) conducted this investigation in two phases. The first phase, a reconnaissance level investigation, included a field inspection of the Mount St. Helens National Volcanic Monument, the Castle Lake debris blockage area and valley sections downstream from the blockage all the way to the SRS. HEC staff attended two days of meetings with project personnel from the Portland District USACE and the U.S. Forest Service to discuss the background of the problem, concerns they and other Federal and State agencies had for the safety of the blockage, and to outline technical procedures for conducting the analytical investigation. The Phase 1 investigation also included a thorough literature investigation and a "reconnaissance-level" (preliminary) mudflow routing investigation. Results from the Phase 1 studies were presented to project managers from the USGS, USFS, State of Washington Department of Ecology and Dam Safety (SWDE) and the Portland District USACE. The results were used to formulate an agreed-upon analytical approach and ranges of breaching and mudflow bulking parameters to be used during the Phase 2 studies. The remainder of this report concentrates on the procedures and results from the Phase 2 investigation.

The Phase 2 investigation included the development of energy based procedures for bulking and debulking the dam break flows from hypothetical breaching of the Castle Lake blockage. The National Weather Service's BREACH model (Fread, 1989) was used to develop breach outflow hydrographs for several types of breaching scenarios, and various lake levels. BREACH is a physically based model that uses soil properties, sediment transport functions, and hydraulic computations to predict the breach characteristics and the discharge hydrograph emanating from a breaching earthen dam or debris blockage. The critical breaching time (defined as the time from the beginning of a major rise in the outflow hydrograph until the time of the peak flow out of the breach) for piping and heave failures was determined to be approximately 15 minutes. Hydrographs were developed for three different initial water surface elevations in Castle Lake: (1) 2,580 feet above NGVD, (2) the lake lowered 30 feet to 2,550 NGVD and (3) the lake lowered 60 feet to 2,520 NGVD. The National Weather Service's

DAMBRK model (Fread 1989) was used to route the dambreach hydrographs down valley, through the Sediment Retention Structure (SRS), and continuing down to the Columbia river. Energy based procedures were developed to simulate the bulking up of the flows via a series of lateral inflow hydrographs. The hydrographs were shaped and positioned along the routing reach so as to provide the appropriate timing and volume of the lateral inflow according to the magnitude of the primary flood wave in the channel. Breakout hydrographs and associated lateral bulking hydrographs were developed for three initial lake levels in Castle Lake and for two different breaching scenarios: (1) a piping failure due to heave as per the USGS's report by Laenen and Orzol (1987), and (2) a piping failure positioned over the historical South Fork Castle Creek outlet channel (referred to as the HEC Breaching Scenario). Downstream bulking of the flows depends on the initial volume and duration of the outflow hydrograph, on the breaching mechanisms and on the valley soil properties and water content. Soil samples were collected from the downstream valley debris deposits by the USGS (Meyer and Dodge, 1988) and the Corps of Engineers (USACE, 1984 and unpublished data, USACE, 1990). A range of measured values for the key parameters used to determine bulking and mudflow characteristics, such as porosity, percent saturation, and expected sediment concentrations, were developed. A Monte Carlo weighting technique (Schaefer 1990) was utilized to determine the most probable combination of these parameters. From the results of the Monte Carlo simulations, high, medium, and low Bulking Factors were selected as a range of probable values for the sensitivity analysis that was conducted by HEC.

Final breaching and routing simulations were conducted based on what is referred to throughout the remainder of the report as "the HEC Breaching and Bulking Scenarios." They represent the breaching and bulking characteristics recommended and agreed upon by the Corps of Engineers and the U.S. Forest Service.

2.1 Physical Setting

The South Fork of Castle Creek is a perennial stream that drains an area of 2.5 square miles on the northwest flank of Mount St. Helens. The Castle Lake blockage is located at the confluence of South Castle Creek and Castle Creek, a tributary to the North Fork Toutle River, approximately 48 and 60 miles upstream from the communities of Castle Rock and Longview-Kelso,WA, respectively. Castle Creek was blocked by a debris avalanche that occurred during the May 18, 1980 eruption of Mount St. Helens, WA. Figures 2 and 3 show the approximate pre-eruption and post-eruption topography of the Castle Lake study area. Avalanche materials formed a blockage approximately 2,000 feet long at the crest and is bounded by bedrock ridges on either end and averages about 1,400 feet wide from lake shore to the downstream toe. Figure 4 shows a typical cross section taken through the debris blockage. The blockage has a maximum height of 190 feet measured from the crest to the toe and 80 feet from the crest to the lake surface.

The blockage consists of two major lithologic units which the USGS refers to as the ancestral dacite unit and the modern dacite, andesite, and basalt unit. The modern (1980 eruption) materials are unsorted, mostly unstratified mixtures of avalanche materials, ranging in size from silt- and clay-sized particles to large clasts more than 5 feet in diameter. Slopes from the crest toward the lake are uniform and average 1V on 4H. Slopes from the crest toward Castle Creek are more varied and range from 1V on 10H to 1V on 2H, with the steepest downstream slopes on the western edge of the blockage. Vertical thickness of the debris blockage ranges from 0 to 250 feet and averages more than 50 feet thick. Location of the deepest zone of avalanche materials corresponds to the former location of the pre-eruption South Fork Castle Creek alignment. It is believed that the old South Fork Castle Creek alignment resulted from breaching and erosion of prehistoric avalanche deposits that formed there during an earlier eruption and blockage sequence that occurred some 2,000 to 3,000 years ago. The original surface of the prehistoric valley deposits was eroded except for the flat swampy area referred to as Castle Creek Marsh in Figure 2. The marsh may have been a remnant from the former prehistoric Castle Lake bed.

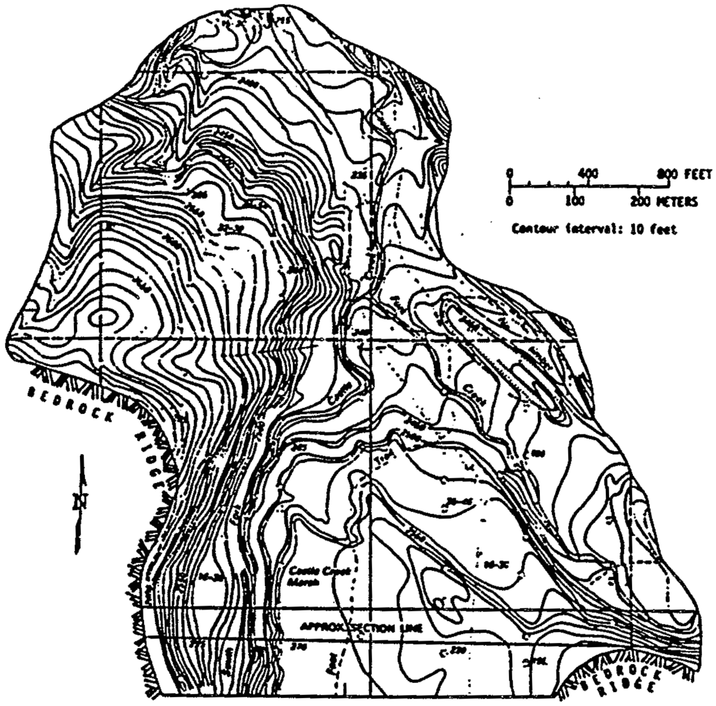


Figure 2
Pre-Eruption Topography of the Study Area

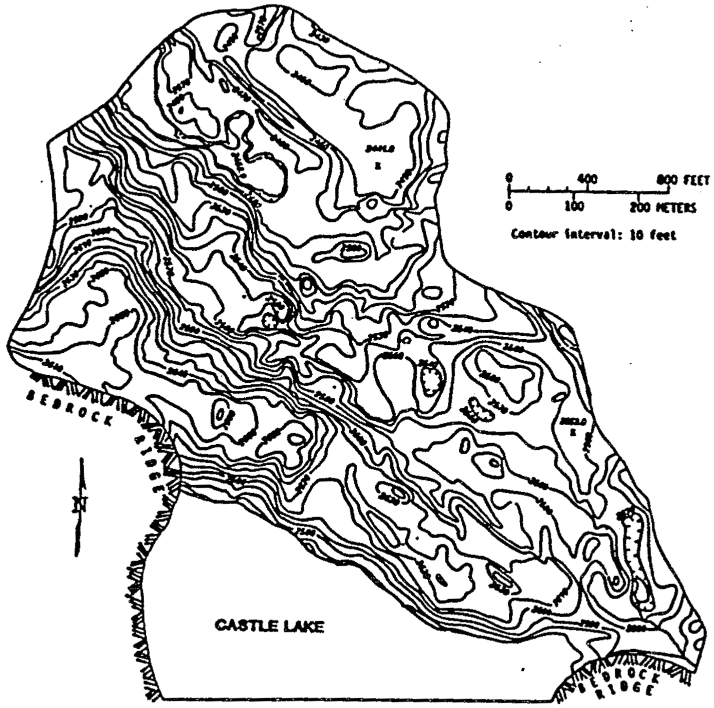


Figure 3
Post-Eruption Topography of the Study Area

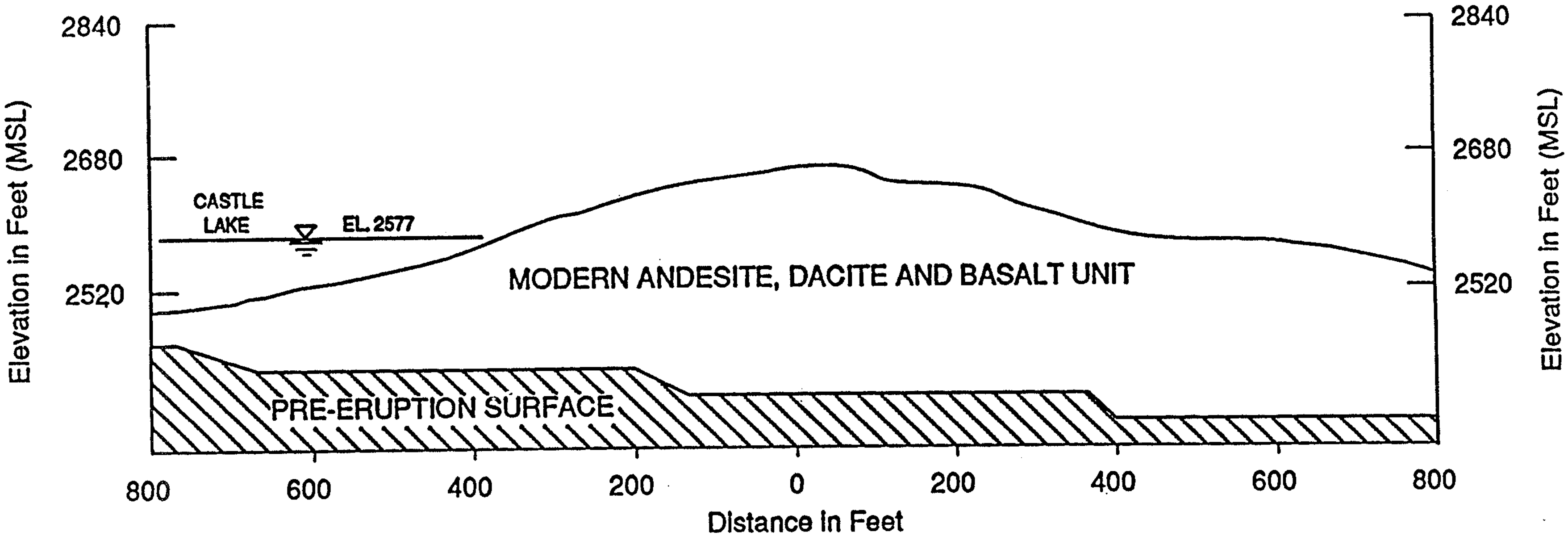


Figure 4
Generalized Geologic Section Through the Castle Lake Debris Blockage

Following the 1980 eruption, a new lake began forming directly behind the debris blockage materials and attained a volume of approximately 19,000 acre-feet before an emergency spillway could be constructed by the Corps of Engineers in 1981 to prevent possible overtopping. Installation of the spillway at the eastern edge of the blockage stabilized the lake elevation at 2,577 feet above NGVD (see Figure 4). At this elevation, the maximum depth in the lake is 110 feet deep and contains approximately 18,500 acre-feet of water.

2.2 Characteristics of Landslide Dams

According to Costa and Schuster (1986) landslide dams form in a wide range of physiographic settings. The most common types of mass movements that can form landslide dams include soil slumps and slides; mud, debris and earth flows; and rock and debris avalanches such as those that occurred during the 1980 Mount St. Helens eruption. The most common initiation mechanisms for potential dam-forming landslides are excessive rainfall, rapid snow melt, earthquakes and volcanic eruptions.

Figure 5 shows that most landslide and debris blockage dams are very short lived. Costa and Schuster (1986) report that for the 63 documented cases they studied, 22 percent of the landslide dams failed in less than 1 day after formation and that half failed within a period of 10 days. Less than 10 percent of the natural debris blockage dams last more than 1 year. They also report that the most frequent mode of failure with debris blockage dams is by overtopping. Figure 6 is adapted from Costa and Schuster, 1986 and shows that more than 50 percent of the documented debris and landslide dams failed due to overtopping. The occurrence of a particular dam failure and the magnitude of resulting floods are predicated by the size of the blockage, its geometric characteristics (size and depth of the impoundment, and size and shape of the blockage), the properties of the blockage materials, the rate of filling of the impoundment, the volume of the trapped water, bedrock or engineered controls such as spillways, tunnels and diversions.

The Castle Lake blockage was ten years old in May, 1990 and appears to be stable under its present conditions. Groundwater levels in the blockage and seeps along the downstream face of the blockage have been monitored since the eruption. According to the Corps' Geotechnical Branch (personal communication, 1990), they have seen no field evidence of unstable conditions in the blockage materials since the installation of the spillway. The Corps of Engineers "Engineering Analysis and Alternative Evaluation" report (1988) concludes that: (1) the risk associated with a single event leading to the failure of Castle Lake in its existing condition is low; (2) the existing blockage is significantly larger than the "minimum embankment section" necessary to safely retain Castle Lake; (3) local areas of instability exist within the blockage, however, these areas are outside the minimum embankment section; (4) no realistic failure mechanism or scenario could be developed that would lead to the sudden, catastrophic release of Castle Lake based upon assumed parameters (Due to the large number of variables involved and uncertainties associated with each, however, it is not possible to completely eliminate all risk), and (5) the blockage exists in an environment where rapid changes are possible (erosion, earthquakes, floods, volcanic eruptions, etc.). Monitoring and maintenance are necessary to ensure that the design assumptions that led to the above conclusions remain valid.

The investigation reported herein is intended to estimate the potential for flooding downstream from the Corps' Sediment Retention Structure for various hypothetical lake breaching scenarios. It is not the intent of his investigation to evaluate any aspect of the risk of failure. It merely quantifies the downstream flood potential for various hypothetical breaching scenarios.

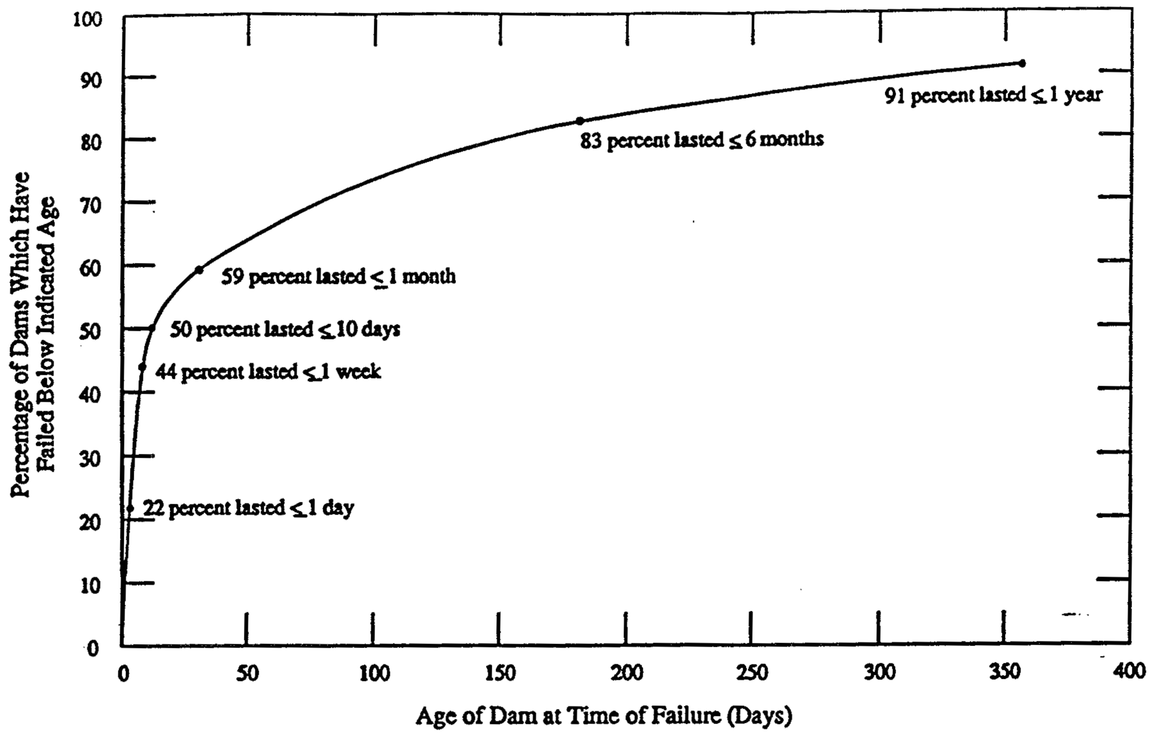


Figure 5
Length of Time Landslide Dams Survive, Based on 63 Cases from the Literature (Adapted from Schuster, 1986)

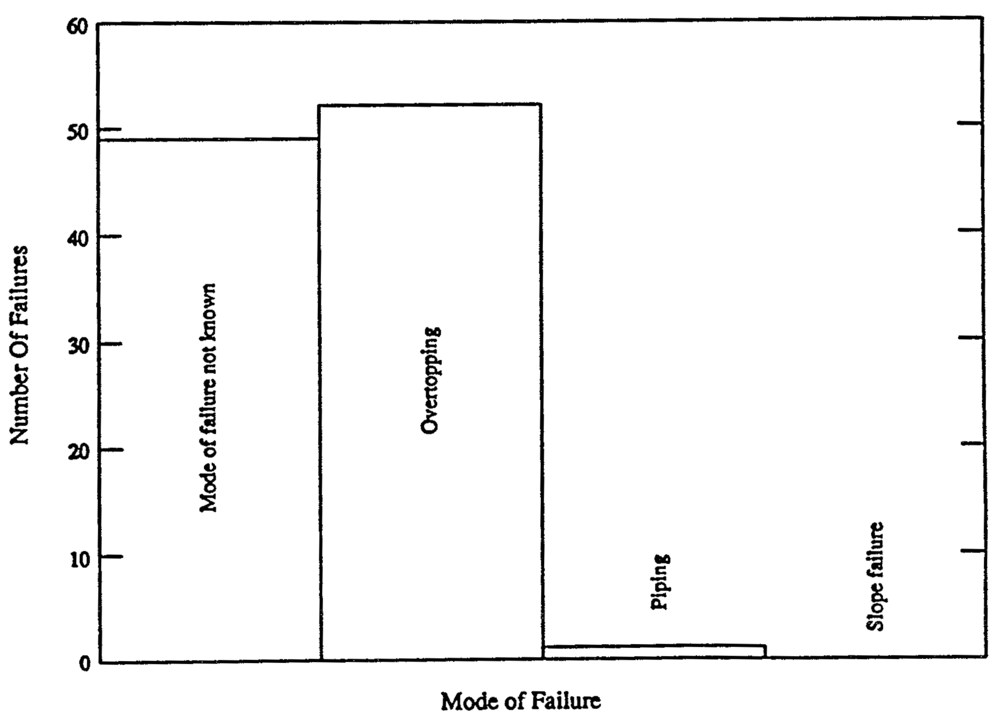


Figure 6
Modes of Failure of Landslide Dams, Based on 103 Cases from the Literature (Adapted from Schuster, 1986)

2.3 Information and Data Sources

Data collection for the Phase 1 portion of this investigation began with a field investigation of the Castle Lake blockage and the downstream channels in November 1989. The field investigation provided a realistic perspective on the characteristics of the debris blockage, the location of the historical South Fork Castle Creek, the characteristics and physical features of the downvalley deposits and the amount of sediment and debris available for flow bulking during high flow events. Many photographs were taken of the channel and valley sections along Castle Creek below the blockage. Manning's n-values were estimated for different sections along the channel and overbanks. Detailed maps and aerial photographs were obtained from the Portland District USACE and from the U.S. Forest Service. The maps and aerial photos cover the area from the Castle Lake debris blockage down to the SRS. Surveyed cross sections were also obtained from the Portland District USACE and from the USGS Open-File 87-549 by Meyer and Dodge (1988). These data were used to develop 47 cross sections between Castle Lake and the SRS and 90 cross sections from the SRS to the Columbia River. The cross sections describe the channel and valley morphology required by the unsteady flow routing model developed for the study reach from Castle Lake to the SRS and from the SRS to the Columbia River. Dimensions and detailed hydraulic information about the Sediment Retention Structure (SRS) and its spillway and outlet works were taken from the Corps' final design manuals for the structure. These data included volume-elevation information, recently surveyed sediment levels behind the structure, and elevation-outflow information for the low flow conduits and emergency spillway.

During the course of this study, it was determined that updated soils information was required to properly estimate the flow bulking potential that exists in debris deposits downstream from Castle Lake. The Portland District sent a soils investigation team to collect soil samples and to measure in situ material properties along the channel and overbank areas below Castle Lake. Data were also collected along the debris blockage itself. This information was used to determine the breaching characteristics (critical time to breach and ultimate breach dimensions) for the debris blockage. These data were also used to estimate the range of possible bulking factors that could occur downstream from the blockage due to various breaching scenarios.

Information and data used during this investigation and presented in this report were obtained from reports, papers and materials provided to HEC by the Portland District Corps of Engineers, the U.S. Forest Service, the U.S. Geological Survey and the Washington State Department of Ecology.

2.4 Breaching Characteristics of the Debris Blockage

Figures 5 and 6 show that most debris blockage lakes fail within one year of their formation and the most common failure mechanism is by overtopping. By installing an emergency spillway in 1981, the Corps of Engineers essentially eliminated the possibility of an overtopping failure of the Castle Lake blockage. Under present conditions, failure of the debris blockage would most likely occur due to a "piping type failure," or as a result of an earthquake occurring in conjunction with a severe hydrologic event that may lead to a "heave type failure" (Laenen and Orzol, 1987). One of the first tasks of the phase 2 portion of this investigation was to estimate the range of possible breach sizes and critical breach times. Three different methods were used to determine possible breach sizes and times. The first two methods are statistically derived regression equations, formulated by MacDonald and Langridge-Monopolis (1984) and by Froelich (1987). Both sets of equations are based on actual data from dozens of historic dam failures. The MacDonald and Langridge-Monopolis study was based on data from 42 man-made earth and rockfill dams (30 earthfill and 12 that were a combination of earth, clay cores, rock fill, and concrete faces). The Froelich study included data from 43 man-made and landslide formed earth dams. Both studies resulted in a set of graphs and equations that can be used to predict the approximate size of the breach and the time it takes for the breach to reach its full failure size.

The third approach for estimating breaching characteristics of the debris blockage was a physically based computer model called BREACH, developed by Dr. Danny Fread (1989) of the National Weather Service. The breach model uses sediment transport and hydraulic routing equations to simulate the formulation of either a piping or over-topping type of failure. The BREACH computer model requires information about the physical dimensions of the dam, as well as a very detailed description of the soil properties of the dam or blockage materials. Required soils information included:

1. D50 (mm)
2. Porosity
3. Unit Weight (lb/ft^3)
4. Internal Friction Angle
5. Cohesive Strength (lb/ft^2)
6. D90/D30

These parameters can be specified separately for the inner core and outside bank materials of a dam. In the case of the Castle lake blockage, the inner core material was assumed to be the same as the outer banks. For this study, the parameters were calculated from soil samples taken by the USGS and the Portland District of the Corps of Engineers. A range of appropriate values was extracted from the field data. A sensitivity analysis was performed to see if the BREACH model would predict different breach sizes for different combinations of the parameters. The sensitivity analysis showed that the size of the breach did not vary significantly over the range of parameters extracted from the field data. Table 1 shows the range of values used in the sensitivity analysis, and the final set of values used for this study.

Table 1. Major soil properties used in BREACH model.

PARAMETER	RANGE OF VALUES	VALUE USED
1. D50 (mm)	1.0 - 9.0	1.0
2. Porosity	.34 - .40	.38
3. Unit Weight (lb/ft^3)	100 - 145	125
4. Internal Friction Angle	34 - 36	35
5. Cohesive Strength (lb/ft^2)	1 - 400	200
6. D90/D30	10 - 125	75

The breaching characteristics for each method, along with the USGS heave scenario developed by Laenen and Orzol (1987), are summarized in Table 2. Also shown are the resulting clear water peak flows that would occur for the respective breaching scenarios. Costa and Schuster (1988) developed a set of curves showing the potential energy of the lake water versus peak discharge from historical dam failures of various types of dams. Figure 7 presents the five different curves developed by Costa and Schuster (1988) for (1) constructed dams, including earth and rockfill dams, (2) landslide dams, (3) Moraine dams, (4) Glacier dams, and (5) an upper envelope curve for all of the observed dam failure data. Table 2 lists the peak discharge estimated from the Costa and Schuster envelope curve (566,000 cfs) using the physical characteristics of Castle Lake. The U.S. Bureau of Reclamation (1977) prepared an earlier curve of observed peak discharge versus the hydraulic depth of a dam prior to failure. Figure 8 presents the U.S. Bureau of Reclamation's curve and shows that for an initial depth of water behind a full Castle Lake, the estimated peak discharge would be approximately 370,000 cfs. The clear water peak flows predicted by all of six of these different methods does not account for the inclusion of sediment from the breach.

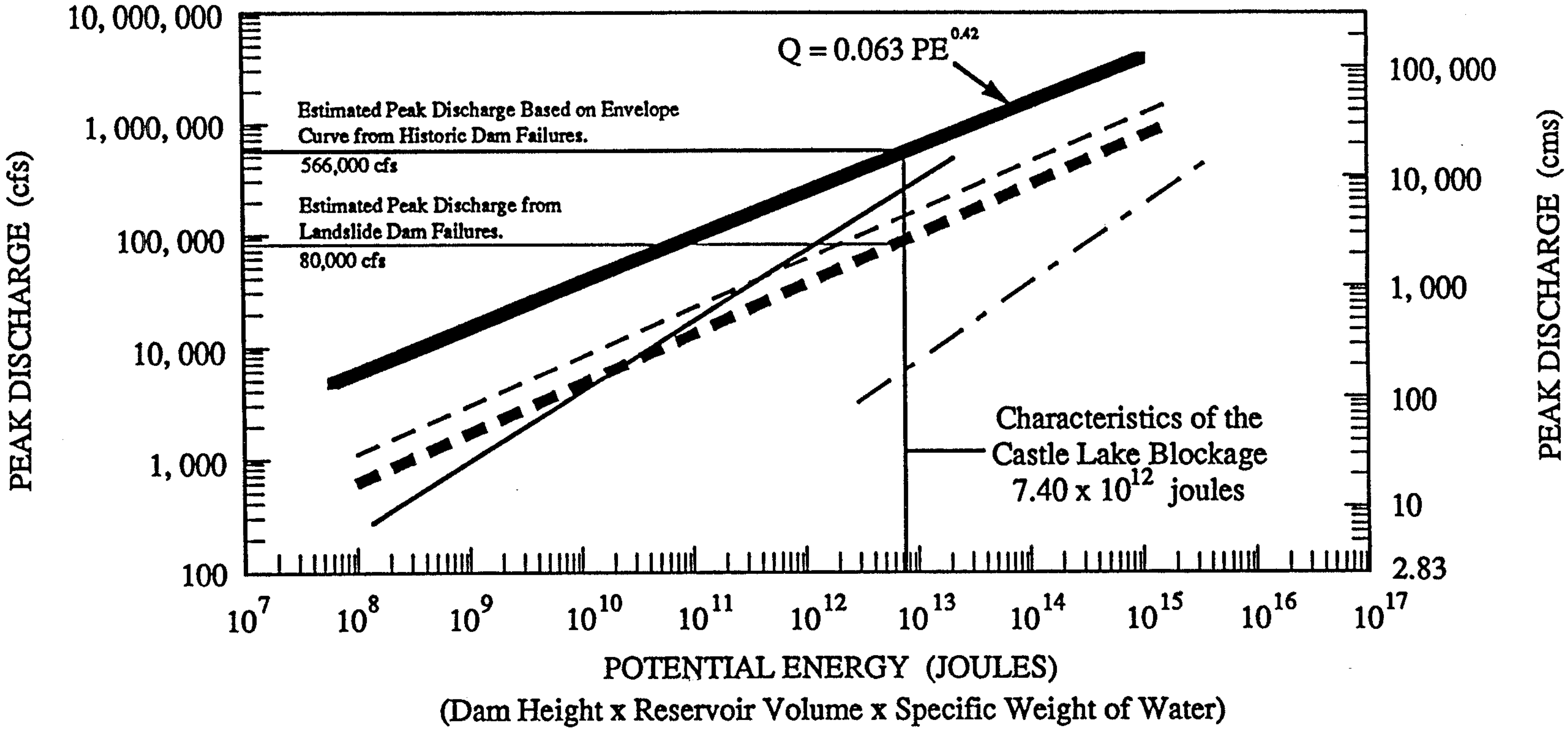
Table 2

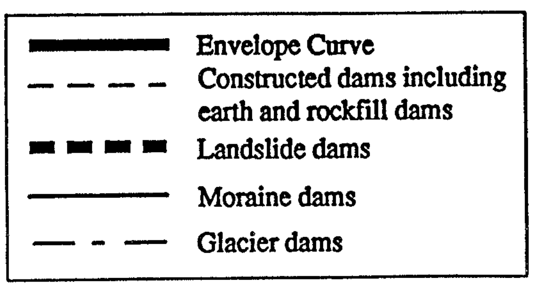
Summary of Breaching Characteristics

BREACHING METHOD	BOTTOM WIDTH (FT)	SIDE SLOPES (H/V)	CRITICAL BREACH TIME (HOURS)	PEAK FLOW FROM CASTLE LAKE (CFS)
U.S.G.S HEAVE[1]	675	1.0	0.25	1,510,000
BREACH MODEL[2] (HEC SCENARIO)	480	0.31	0.25	1,180,000
FROELICH[3] EQUATIONS	305	0.31	0.36	761,300
POTENTIAL ENERGY[4] VERSUS PEAK Q RELATIONSHIPS - HISTORICAL DATA	-	-	-	566,000
U.S.B.R CURVE - [5] HISTORICAL DATA	-	-	-	370,000
MacDONALD[6] LANGRIDGE-MONOPOLIS	25	0.31	0.50	147,600

1 Laenen and Orzol (1987)
2 HEC's Breaching Scenario using the NWS BREACH model (1989)
3 Froelich (1987)
4 Costa and Schuster (1988)
5 U.S. Bureau of Reclamation (1977)
6 MacDonald and Langridge-Monopolis (1984)

Figure 7
Potential Energy Versus Peak Discharge Relationships for Various Types of Dam Failures With Estimated Peak Discharges for the Castle Lake Blockage
(adapted from Costa & Schuster, 1988)





The dashed lines are least-square regression lines for different kinds of dams. They represent the "most likely" estimate of peak discharges for different types of dams. The larger top line is the envelope curve for all dam-failure data. It represents a "conservative" peak discharge based on known historic failures of all types of dams.

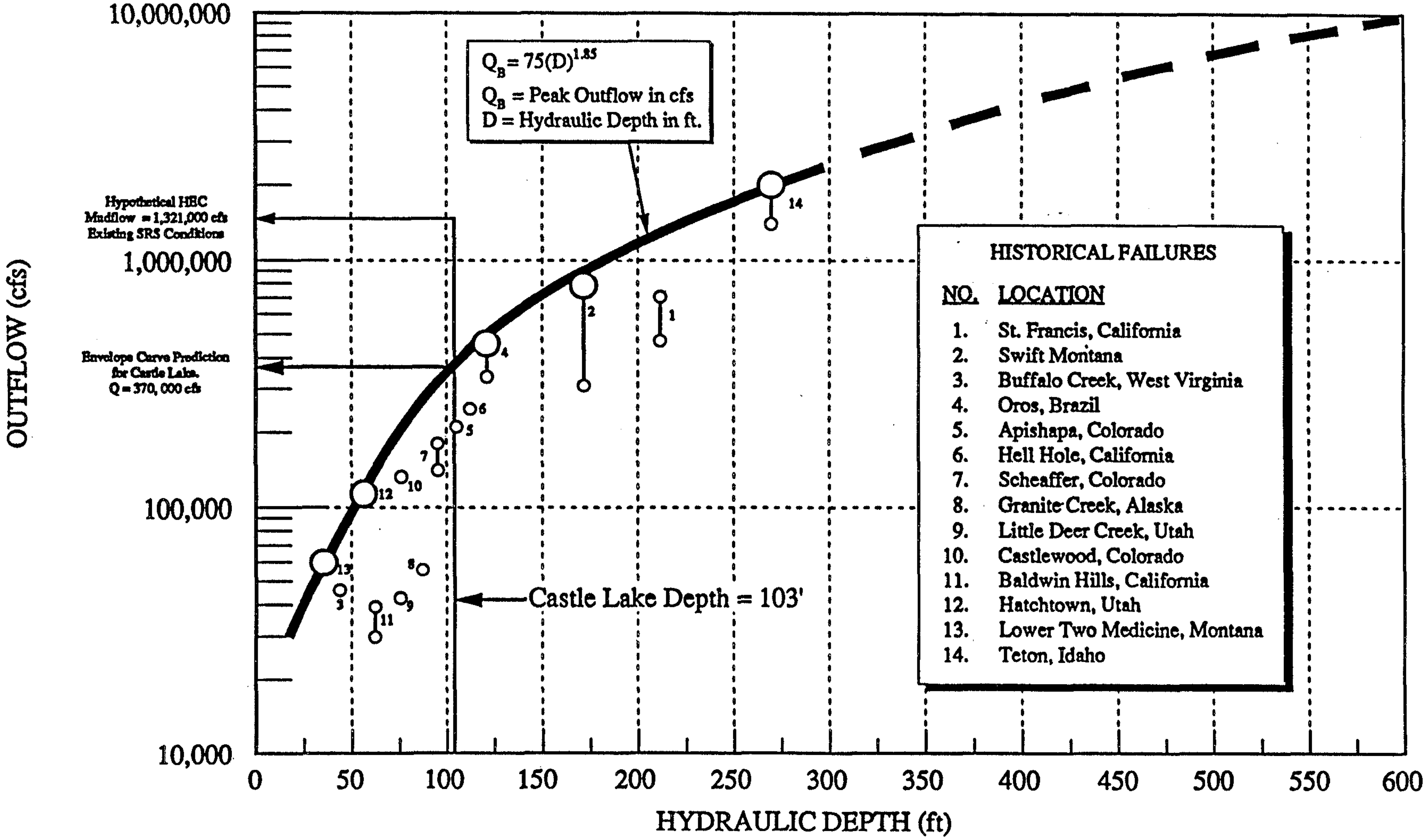


Figure 8
Envelope Curve of Discharges Experienced from Historical Dam Failures
(from U.S. Bureau of Reclamation, 1977)

After examining the physical characteristics of the Castle Lake blockage and comparing the breaching characteristics and peak discharge estimates from the various breach development methods listed in Table 2, results from the BREACH model are thought to be the most reliable and his method for estimating the breaching characteristics of Castle Lake the most dependable. The MacDonald and Langridge-Monopolis results are considered inappropriate because their data set did not include any landslide or debris blockage failures. Results from the BREACH model were selected over the Froelich equations because the BREACH model is a more physically based method and it accounts for the material characteristics of the blockage more explicitly, therefore providing a better representation of the specific problem at Castle Lake. Results from the HEC breaching scenario (using the BREACH model) are more conservative than those from other methods.

Dozens of different types of failure scenarios were tested to try to duplicate the characteristics of the USGS proposed "heave type" failure. No reasonable set of failure parameters could reproduce a breach with a top width of 1000 feet and an approximate depth of 175 feet within the critical breach time of 15 minutes. HEC even tried to simulate the retrogressive heave failure mechanism by initially removing 65 percent of the thickness of the blockage materials from the downstream face of the blockage, and then starting the BREACH model in a piping mode with a full lake. Even under these simulated heave failure conditions, the resulting breach size was similar in overall dimensions to those produced by the "HEC breaching scenario." After considerable discussion with engineers and geologists from the Corps of Engineers and the U.S. Forest Service, it was decided to adopt HEC's proposed breaching scenario using the BREACH model as the most representative breaching approach for the remainder of the investigation.

2.5 Flow Bulking and Mudflow Routing Procedures

The BREACH model was used to generate the dambreak outflow hydrographs from Castle Lake for various initial lake levels and breaching scenarios. Routing of the bulked dambreak hydrographs (mudflows) 65 miles from Castle Lake down the North Fork Toutle and Cowlitz Rivers to the Columbia River was accomplished with Fread's (1989) DAMBRK model. Two separate routing reaches were established as shown in Figure 9. The first reach (routing reach 1) extends from Castle Lake, 16 miles down to the SRS. Flow bulking and debulking processes occur within this reach (see Laenen and Orzol, 1987). The second routing reach (routing reach 2) extends from the SRS all the way to the Columbia River below Kelso-Longview. The "mudflow routing option" in the DAMBRK computer program was used to simulate the non-Newtonian hyperconcentrated flow properties of the bulked discharges in routing reach 1 downstream from Castle Lake. The program requires the user to specify the expected mudflow properties, such as viscosity and initial shear strength of the fluid. Based on these expected fluid properties and the hydraulic characteristics of the flow, the "mudflow routing option" adjusts the effective friction loss terms in the momentum equations to simulate the effects of hyperconcentrated (bulked) flow.

Flow bulking is the process whereby extremely high energy flood flows incorporate additional bed material (sediment and debris) into the flow by erosion and entrainment, thus increasing the total volume of the flood. As the concentration of suspended material increases beyond a threshold of approximately 20 to 60 percent by volume (Beverage and Culbertson, 1964), the fluid-sediment mixture begins to demonstrate non-Newtonian fluid characteristics (i.e., the flow characteristics become dependent upon the concentration of suspended material). Many researchers have estimated that flow bulking can increase the overall volume of a hypothetical dambreak from Castle Lake from 2 to 5 times its original volume (Costa, 1984, Laenen and Orzol, 1989, and Scott, 1988). The actual amount of flow bulking that occurs during an event depends on many factors and is difficult to estimate ahead of time. Schaefer (1990) derived a simplified relationship that estimates the "ultimate bulking factor" (BF), given the representative "in situ" soil characteristics of the avalanche materials in the valley and channels downstream from the location of hypothetical lake breakout. Soil porosity or void ratio, along with the percent saturation of the soil are required, along with an estimate of the ultimate sediment concentration by volume that may occur during the bulking process. Shaefer's derivation of the "ultimate bulking factor" is shown in appendix A. Scott (1985a, 1985b), Costa (1984), Schuster (1986) and Laenen and Orzol (1987) suggest that the suspended sediment concentrations for this type of an event may range from 45 to 55 percent by volume. Therefore, a value of 50 percent for a representative ultimate concentration seems to provide a reasonable assumption for this study.

Bulking and debulking are likely to occur within reach 1 from Castle Lake down to the SRS according to Laenen and Orzol (1987). They also suggest that bulking will occur from Castle Lake to the N-1 structure and debulking (loss of suspended sediment materials from the flow) from the N-1 to the SRS (see Figure 9). Reach 1 upstream from the N-1 structure, has the deepest avalanche deposits, most erodible channel materials, the narrowest valley sections, the steepest stream slopes, and the greatest sediment transport potential. Debulking is likely to occur downstream from the N-1 structure where the valley widens to several thousand feet, the effective channel slope decreases considerably and the sediment transport capacity decreases below that necessary to sustain the high concentrations of materials entrained from bulking.

Bulking of the dambreak flows up to concentrations of 50 percent by volume as suggested by Laenen and Orzol (1987) was simulated by adding a series of lateral mudflow hydrographs to the main dambreak hydrograph as it was routed downstream from Castle Lake. In this way, the effects of flow bulking are essentially "blended into the main dambreak hydrograph" as it moves down valley. This process was accomplished by first calculating the potential bulking factor for each breaching scenario using Schaefer's (1990) method. Clear water dambreak hydrographs from Castle Lake for the various breaching scenarios were then routed down to the SRS with no accounting of the potential bulking or debulking processes. Hydrographs at several points along bulking reach 1 (see Figure 9) were extracted from the clear water runs. These hydrographs were then multiplied by a ratio [the estimated subreach Bulking Factor (BF)] to account for the bulked volume of material that would need to be added to each subreach to obtain the overall flow bulking and total flow volume for the entire bulking reach for each breaching scenario. Distribution of the amount of bulked material entering the flow with distance in reach 1 was allocated according to the longitudinal distribution of sediment transport capacity along the bulking reach. This method produces a more realistic (nonlinear) relationship between local hydraulic conditions (depth, width and velocity) and the longitudinal change in the amount of bulked flow to be blended into each subreach.

The tendency for flow bulking actually increases for a short distance downstream from the blockage because the valley is relatively narrow and very steep and the transport capacity is very high along the front of the dam break bore. Consequently, as the flow picks up more material from bulking processes, the effective discharge also goes up until the valley widens and flattens enough to begin attenuating the flow. This phenomenon of the magnitude of the flow increasing due to bulking for some distance downstream from the initial breakout is clearly demonstrated in the results shown and discussed in later sections of this report. As the valley widens and the channel slope decreases, the amount of bulking goes down in proportion to the reduced transport capacity for that subreach. This approach was used to distribute the amount of bulked material (nonlinearly) into the flows from the dam break hydrograph as it was routed dynamically downstream from Castle Lake to the N-1 structure. The last step was to run the model again using the "mudflow option" with the estimated mudflow properties for each reach, while adding (blending) the lateral bulked flow hydrographs to main flow to account for the dynamic flow bulking.

Laenen and Orzol, (1987) suggest that debulking occurs in lower portion of reach 1, from the N-1 structure to the SRS. The valley widens rapidly along this part of the reach to several thousand feet wide, the stream slope decreases, and there is insufficient sediment transport capacity to sustain continued bulking of the flows. The debulking process is handled in a similar manner as was the bulking process. Hydrographs from the clearwater runs for the different breaching scenarios are used to establish the shape and timing of lateral flows that would be extracted (debulked) from the main flood wave. Once again the distribution of the amount of debulking is allocated along the debulking reach in proportion to the change in the amount of sediment transport capacity occurring along the reach. Debulking is simulated with the DAMBRK model by using negative lateral flow hydrographs to remove the amount of flow volume lost due to debulking. Laenen and Orzol (1987) used similar methods to simulate the expected debulking below the N-1 structure. This process was repeated for each breaching scenario that was evaluated.

2.6 Estimating the Ultimate Bulking Factor for Various Dambreak Scenarios

The magnitude of the ultimate bulking factor is not only a function of the characteristics of the dambreak hydrograph producing the flow, but also a function of the in situ properties of the valley and channel deposits where the dambreak flows will occur. Examination of field data collected by the Corps of Engineers and the USGS, shows that actual field values for porosity and void ratio, percent saturation and ultimate suspended sediment concentration can vary according to the season, material type and location along the channel. For the purposes of this investigation, a method was derived to bracket the range of possible material properties observed in the field and to assign a level of confidence to the many possible combinations of the three main variables (porosity, percent saturation and ultimate suspended sediment concentration) that can occur. For this study, the following range of values for the main parameters was agreed upon:

1. Porosity (0.25 - 0.45)
2. % Saturation (45 - 90)
3. Ultimate concentration (0.30 - 0.60)

Schaefer (1990) developed a statistical weighting method using Monte Carlo sampling techniques, to estimate the magnitude of the "ultimate bulking factor" according to observed ranges in the magnitudes of the three main variables used to compute the bulking factor (BF). The Monte Carlo method computed a "cumulative probability" of 99.7% to the USGS' "heave scenario with a bulking factor of 4.46." The method assigns a 95% cumulative probability to the HEC "piping scenario with a BF = 3.32," and 50% to the HEC "piping scenario with a BF = 2.5." The cumulative probability means that of all the possible bulked flows that can occur during a breaching of Castle Lake with the range of observed material properties in the channel downstream from the blockage, 99.7, 95, and 50 percent of the events will have bulking factors less than 4.46, 3.32 and 2.5, respectively. Therefore, of all the different combinations of values of the three bulking factor variables, less than 1/2 of a percent of all the possible combinations would yield a bulking factor as large as 4.46 for the USGS' breaching scenario. Approximately 95 percent of all the possible events would be less than the 3.32 bulking factor associated with HEC's piping scenario and about 50 percent of all the events would have a bulking factor of 2.5 or less. This range of bulking factors was used to evaluate the potential for flooding downstream from the SRS for three hypothetical lake breaching scenarios. Results and conclusions from the analyses are presented in following sections of the report.

2.7 Other Boundary Conditions Considered

Figure 9 presents a schematic diagram of the approximate location and extent of the routing reaches considered during this investigation. Three different breaching scenarios were considered for the hypothetical failure of the Castle Lake blockage. Tables 3 and 4 summarize the specific breaching scenarios that were considered along with the boundary conditions and range of bulking factors that were applied. Tables 3 and 4 also list the key locations along routing reaches 1 and 2 where computed results are displayed for comparison and discussion. As discussed in section 2.5, routing of the hypothetical breaching events was carried out in two different reaches. Reach 1 included the flow bulking and debulking effects on the flows from Castle Lake to the SRS. Reach 2 extends from the SRS all the way to the Columbia River. It is assumed that most of the sediment and debris load will be captured behind the SRS and that flow bulking and/or debulking is insignificant throughout reach 2. Tables 3 and 4 show the initial Castle Lake water surface elevations that were considered: (1) lake full at elevation 2,580 NGVD, (2) lake lowered 30 feet to elevation 2,550 NGVD, and (3) lake lowered 60 feet to elevation 2,520 NGVD. The spillway crest elevation at Castle Lake is 2,577 feet NGVD. The starting lake elevation of 2,580 corresponds to the estimated lake level that would occur during a 500 year rainfall event positioned over the 3.0 square mile drainage area above the lake. Under these conditions, a discharge of approximately 6,000 cfs would be flowing out of Castle Creek as base flow prior to the dam break. The estimated runoff and base flow in the North Fork Toutle River upstream from the N-1 structure prior to the dam break was approximately 20,000 cfs. Other assumed base flow conditions include (1) 6,000 cfs from the SRS, (2) 5,700 cfs from the Green River into the North Fork Toutle River, (3) 6,100 cfs from the South Fork Toutle River, and (4) 52,000 cfs in the Cowlitz river just upstream from the confluence with the Toutle River. These base flow conditions were recommended by the Portland District's Hydrology and River Engineering Branch for the purposes of this investigation. The total base flow in the Cowlitz river downstream from the confluence with the Toutle River as a result of the cumulative base flows from the rivers entering above is approximately 69,800 cfs.

The other key boundary condition considered as a variable during the investigation was the initial storage condition behind the SRS. If an event were to occur today (in September, 1990) the storage conditions would be represented by the "existing conditions" in the SRS (see Tables 3 and 4 for routing reach 2, SRS to the Columbia). However, the SRS is designed to trap sediment materials from Mount St. Helens and the drainages upstream from the SRS. Therefore, over time the amount of available storage will decrease until the SRS is completely full of sediment and debris up to the crest of the spillway. Even under "completely full conditions", the SRS provides a great deal of storage and attenuation of flood waves as will be discussed in the next section.

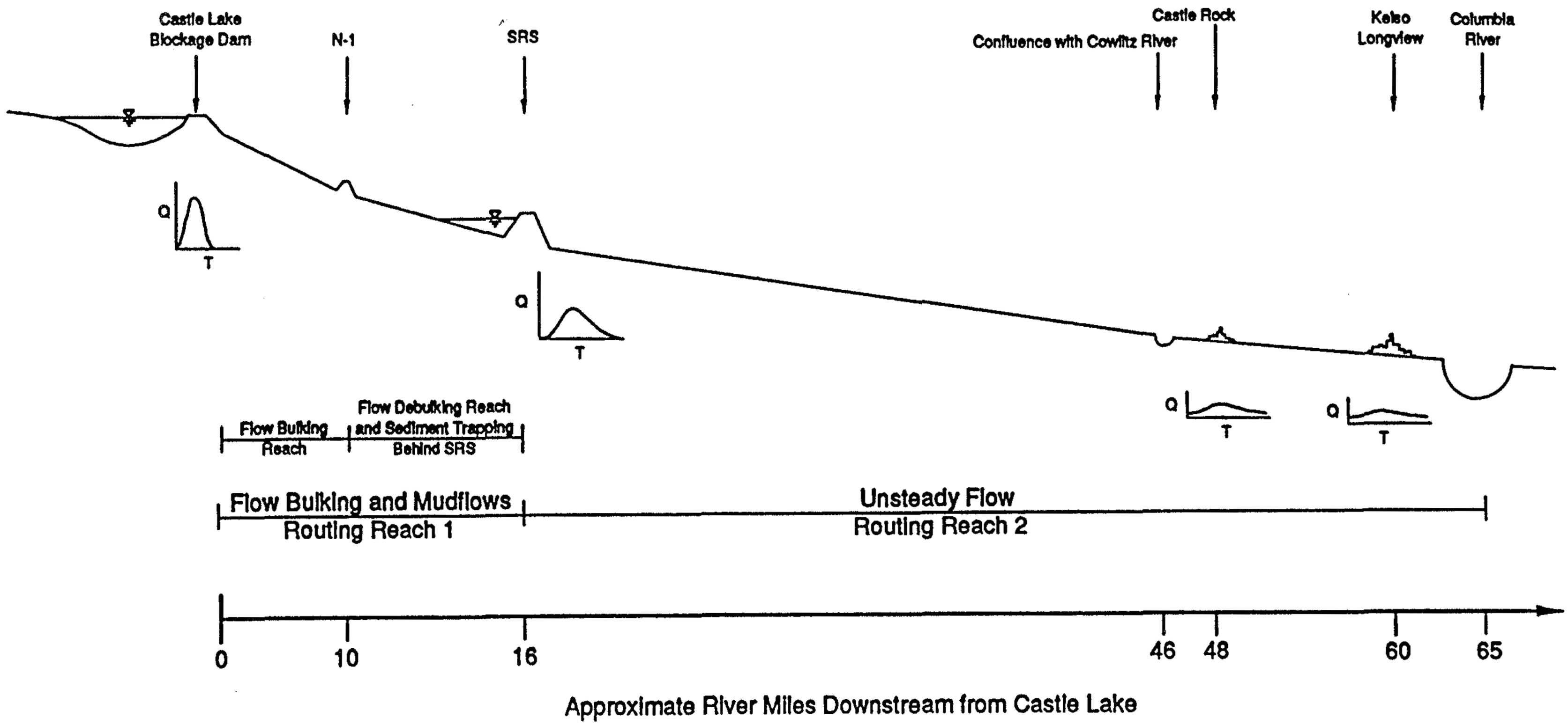


Figure 9

Schematic Diagram of the Flow Bulking, Debulking and Unsteady Flow Routing Reaches from Castle Lake to the Columbia River

TABLE 3

Summary of Simulations Performed and Locations Where Results Were Printed

CASTLE LAKE TO SRS ROUTING REACH

HEC SCENARIO BF=2.50

LAKE LEVEL FULL	LAKE LEVEL −30'	LAKE LEVEL −60'	KEY LOCATIONS WHERE RESULTS WERE PRINTED (MILES D/S FROM CAST. L.)
X			0.0, 1.87, 5.31, 10.19, 12.8, 15.71
	X		0.0, 1.87, 5.31, 10.19, 12.8, 15.71
		X	0.0, 1.87, 5.31; 10.19, 12.8, 15.71

HEC SCENARIO BF=3.32

LAKE LEVEL FULL	LAKE LEVEL −30'	LAKE LEVEL −60'	KEY LOCATIONS WHERE RESULTS WERE PRINTED (MILES D/S FROM CAST. L.)
X			0.0, 1.87, 5.31, 10.19, 12.8, 15.71
	X		0.0, 1.87, 5.31, 10.19, 12.8, 15.71
		X	0.0, 1.87, 5.31, 10.19, 12.8, 15.71

SRS TO COLUMBIA RIVER ROUTING REACH

HEC SCENARIO BF=2.50

LAKE LEVEL FULL	LAKE LEVEL −30'	LAKE LEVEL −60'	SRS FULL	SRS EXIST. COND.	KEY LOCATIONS WHERE RESULTS WERE PRINTED (MILES D/S FROM CAST. L.)
X			X		15.91, 23.21, 33.91, 47.91, 59.81
X				X	15.91, 23.21, 33.91, 47.91, 59.81
	X		X		15.91, 23.21, 33.91, 47.91, 59.81
	X			X	15.91, 23.21, 33.91, 47.91, 59.81
		X	X		15.91, 23.21, 33.91, 47.91, 59.81
		X		X	15.91, 23.21, 33.91, 47.91, 59.81

HEC SCENARIO BF=3.32

LAKE LEVEL FULL	LAKE LEVEL −30'	LAKE LEVEL −60'	SRS FULL	SRS EXIST. COND.	KEY LOCATIONS WHERE RESULTS WERE PRINTED (MILES D/S FROM CAST. L.)
X			X		15.91, 23.21, 33.91, 47.91, 59.81
X				X	15.91, 23.21, 33.91, 47.91, 59.81
	X		X		15.91, 23.21, 33.91, 47.91, 59.81
	X			X	15.91, 23.21, 33.91, 47.91, 59.81
		X	X		15.91, 23.21, 33.91, 47.91, 59.81
		X		X	15.91, 23.21, 33.91, 47.91, 59.81

TABLE 4

Summary of Simulations Performed and Locations Where Results Were Printed

CASTLE LAKE TO SRS ROUTING REACH

USGS HEAVE SCENARIO BF=4.46

LAKE LEVEL: FULL	LAKE LEVEL: -30'	KEY LOCATIONS WHERE RESULTS WERE PRINTED (MILES D/S FROM CAST. L.)
X		0.0, 1.87, 5.31, 10.19, 12.8, 15.71
	X	0.0, 1.87, 5.31, 10.19, 12.8, 15.71

SRS TO COLUMBIA RIVER ROUTING REACH

USGS HEAVE SCENARIO BF=4.46

LAKE LEVEL		SRS		KEY LOCATIONS WHERE RESULTS WERE PRINTED (MILES D/S FROM CAST. L.)
FULL	-30'	FULL	EXIST. COND.	
X		X		15.91, 23.21, 33.91, 47.91, 59.81
X			X	15.91, 23.21, 33.91, 47.91, 59.81
	X	X		15.91, 23.21, 33.91, 47.91, 59.81
	X		X	15.91, 23.21, 33.91, 47.91, 59.81

3. Results and Discussion

Figure 10 presents curves of the computed peak discharge versus river mile resulting from the hypothetical failure of the Castle Lake blockage due to piping for three initial lake levels and two different SRS conditions. The "ultimate bulking factor" for the scenarios simulated and presented in Figure 10 was 2.50. Results for this scenario are considered to be the approximate average (representative of 50 % of the events that may occur) based on the field data presently available. Figure 11 presents similar curves of peak discharge versus river mile for the same type of piping failure but with an "ultimate bulking factor" of 3.32. These results represent a scenario with a mode of failure and a magnitude of bulking that could be equalled or exceeded by only 5 percent of the possible flow events that were considered (or 95 % of the events will be less than this magnitude). This breaching and bulking scenario is recommended by the Corps of Engineers, the U.S. Forest Service and the Washington State Department of Ecology for the evaluating the flooding effects a hypothetical beaching of the Castle Lake blockage. Figure 12 presents similar results for the USGS' "heave type breaching scenario" introduced by Laenen and Orzol (1987) and modified by HEC. For this scenario HEC used Laenen and Orzol's breach hydrograph, but applied HEC's energy related bulking approach along with Schaefer's "ultimate bulking factor" of 4.46 based on the assumed field conditions. Only two initial lake elevations were considered for this scenario. The results are presented in Figure 12 for lake full conditions and the lake lowered 30 feet. The "ultimate bulking factor" associated with the USGS' breaching scenario was 4.46. This represents a cumulative probability (see Section 2.6) of 99.7 percent; e.g. 99.7 percent of the events considered will be less than the computed discharges presented in Figure 12. Table 5 summarizes the routing results at key locations below Castle Lake for the three different breaching and bulking scenarios considered: (1) HEC's piping failure with the median BF = 2.5, (2) HEC's piping failure with the 95 % BF = 3.32 and (3) the USGS' heave failure with the 99.7 % BF = 4.46. The peak discharges listed in Table 5 should approximately equal the peak discharges plotted in Figures 10 through 12. Note that the initial elevation of Castle Lake and amount of sediment stored in the SRS make a considerable difference in the magnitude of the initial dambreak flow and, therefore, a great deal of difference by the time the flood wave is routed to the SRS and downstream to Castle Rock and Kelso - Longview. For example, Table 5 lists the routing results for the Corps'- recommended scenario (runs 7 through 12: HEC's piping scenario with the 95 % BF = 3.32). Note that with Castle Lake full (elevation = 2580 NGVD) and existing at the SRS (see run 7 in Table 5), the peak discharge in the North Fork Toutle River below the SRS is reduced 85 percent (from 695,000 to 105,200 cfs) with the SRS in place. If the lake is lowered 30 feet or 60 feet (runs 8 and 9), the flow reduction due to storage in the SRS is approximately 82 and 95 percent, respectively. Even if the SRS were initially full of sediment (see runs 10, 11 and 12 in Table 5), the peak discharge from a failure of Castle Lake would be significantly reduced. For this case the flows would be reduced by 76, 66 and 56 percent, for the lake full, the lake lowered 30 feet and the lake lowered 60 feet, respectively.

Figures 10 through 12 clearly show the boundary condition effects on the peak discharges for various combinations of initial lake levels, SRS conditions and bulking factor. Because the SRS substantially reduces the flow downstream from the SRS, dambreak discharges added to the base flows in the Toutle and Cowlitz Rivers downstream from the SRS may not be significantly greater than the initially assumed base flow conditions. For the case of runs 7 through 9 (see Figure 11 and Table 5), only 39,600, 28,700 and 400 cfs are added to the base flow in the Cowlitz River at Kelso - Longview due to the dambreak discharges estimated with the initial lake full, lowered 30 feet and lowered 60 feet, respectively. If the SRS were full of sediment at the time of failure (runs 10 through 12), 59,000, 46,000 and 18,600 cfs would be added to the 69,800 cfs base flow in the Cowlitz River at Kelso - Longview. This represents an increase in discharge above the assumed base flow in the Cowlitz River of 85, 66 and 27 percent, respectively for the three different initial lake conditions.

Figure 10
Peak Discharge Versus River Mile Resulting from the Hypothetical Failure of Castle Lake for Three Initial Lake Levels and Two Different SRS Conditions

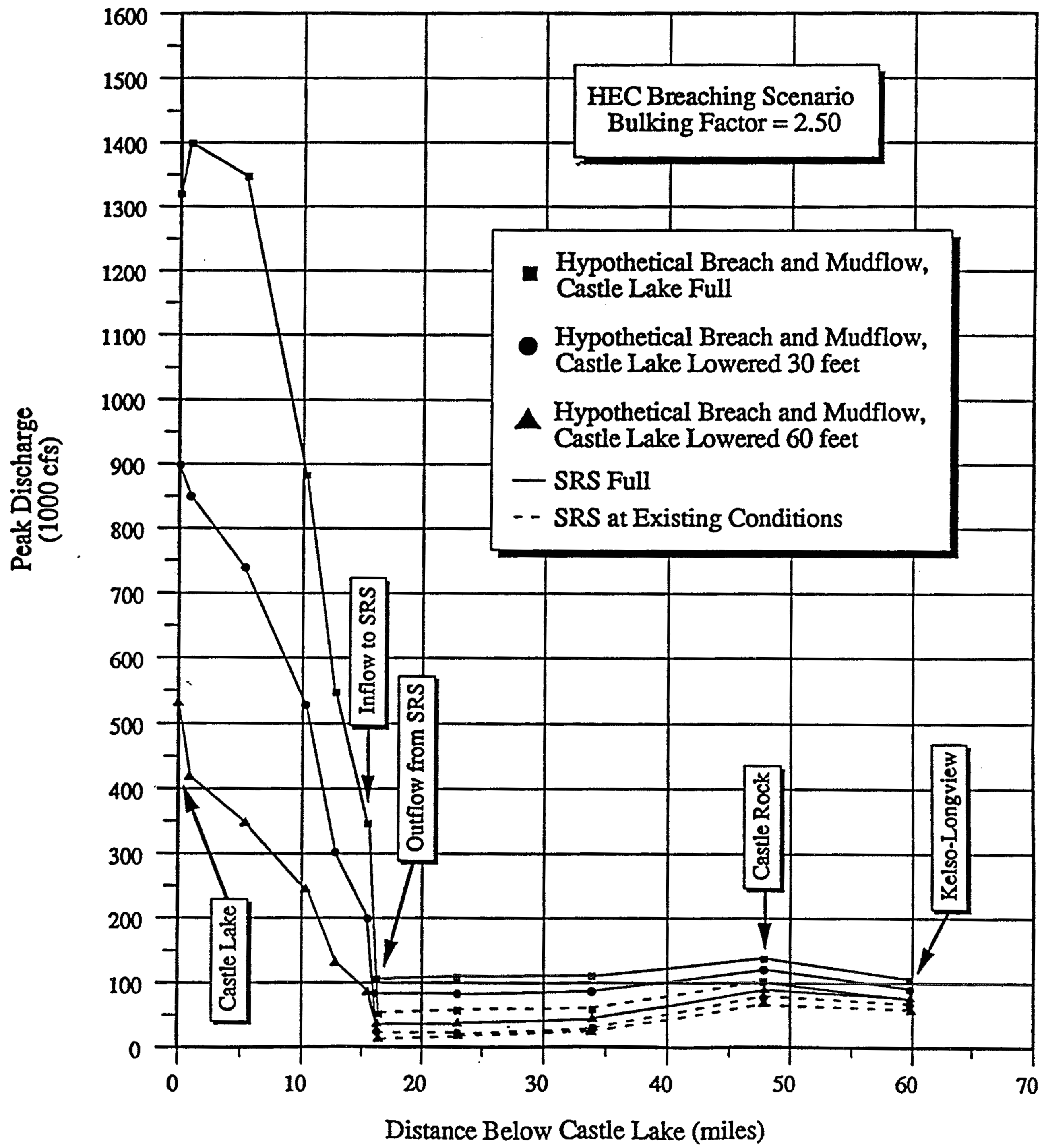


Figure 11
Peak Discharge Versus River Mile Resulting from the Hypothetical Failure of Castle Lake for Three Initial Lake Levels and Two Different SRS Conditions

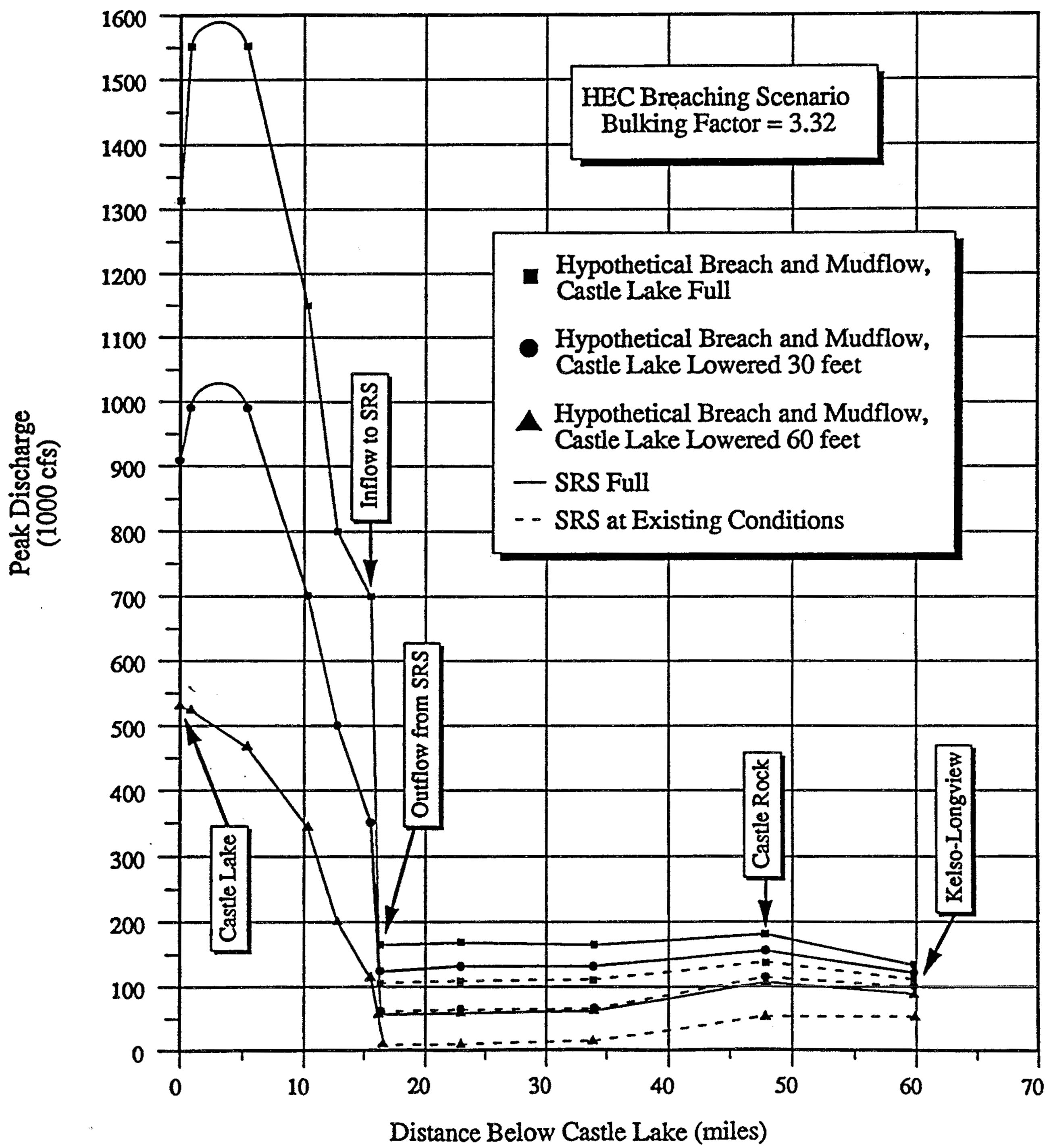


Figure 12
Peak Discharge Versus River Mile Resulting from the Hypothetical Failure of Castle Lake for Two Initial Lake Levels and Two Different SRS Conditions

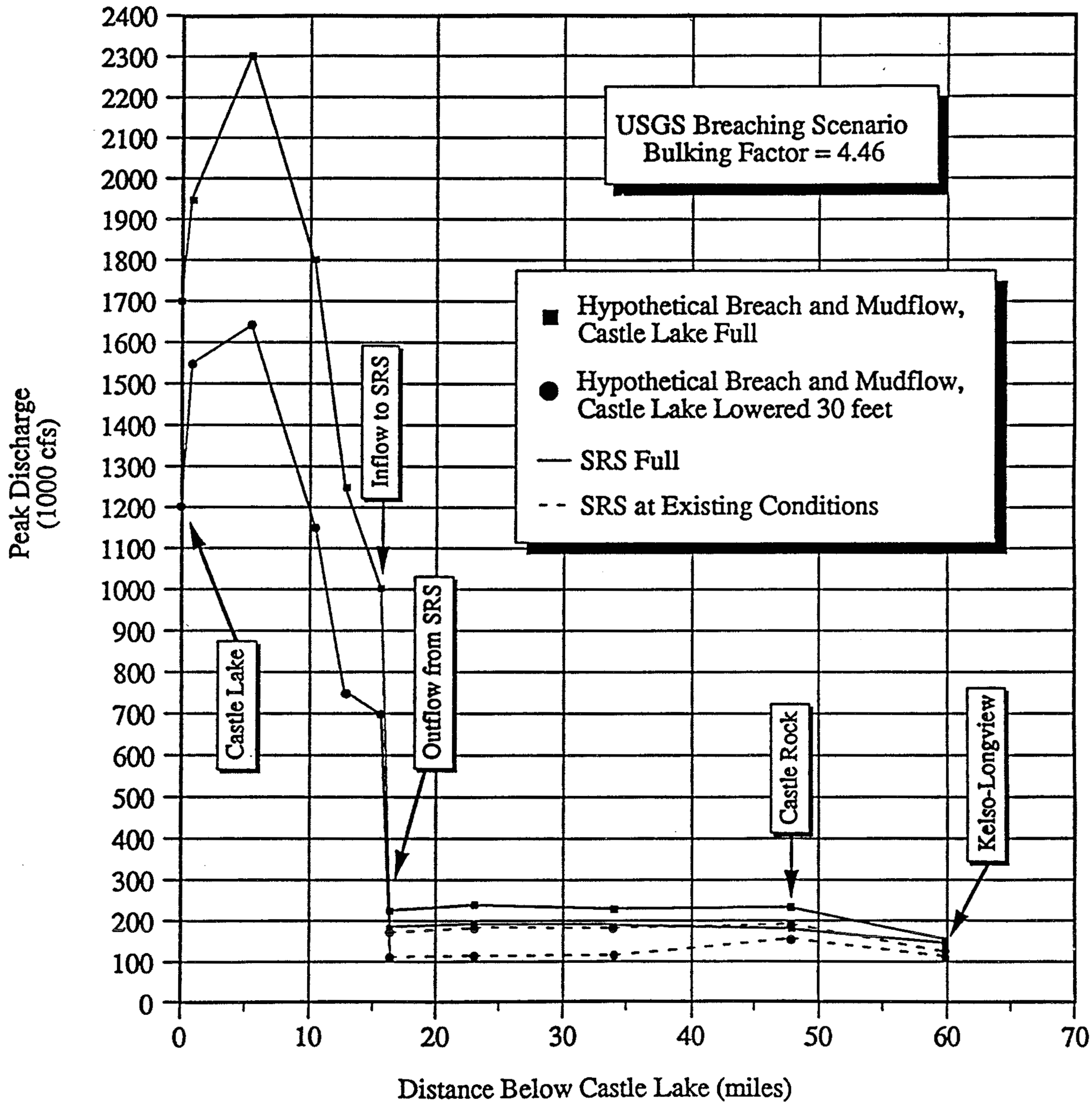


Figure 13 summarizes the estimated flood magnitudes from Castle Lake to Kelso - Longview for the HEC breaching and bulking scenario (BF = 3.32), for two initial lake conditions (full and lowered 30 feet) and existing (1990) conditions in the SRS. The hypothetical HEC mudflow conditions are compared to the 1980 Mount St. Helens mudflow and the probable maximum flood (assuming the SRS was not there during the PMF). The spillway capacity at the SRS is indicated in Figure 13, along with the approximate maximum channel capacities near the communities of Castle Rock and Kelso - Longview. Both hypothetical HEC dambreak floods (for lake full and lowered 30 feet) represent "existing conditions" at the SRS and in the Cowlitz River. The routed dambreak flows for these "existing conditions" are all contained within the present Cowlitz River at Castle Rock and Kelso - Longview. If the SRS were full of sediment initially, the estimated HEC dambreak flood would exceed the present channel capacity near Castle Rock by 28,700 and 11,800 cfs if Castle Lake were initially full or lowered by 30 feet, respectively. The channel near Kelso - Longview (estimated to be 140,000 cfs) presently has sufficient capacity to contain routed dambreak flows, even if the SRS were full of sediment. Estimated peak flows from the lessor HEC scenario (with a bulking factor of 2.5) would all be contained within the present channel at Castle Rock and Kelso - Longview. If an event as rare as the hypothetical "heave failure scenario" (with a bulking factor of 4.46) were to happen under the "existing conditions", the channel capacity near Castle Rock would be exceeded for both initial lake levels that were considered (Castle Lake full or lowered by 30 feet). Flood flows would still be contained in the present channel near Kelso - Longview under "existing conditions." If the SRS were initially full, both Castle Rock and Kelso - Longview could experience flooding for lake full initial conditions. If the lake were lowered 30 feet, Castle Rock could still experience flooding, while the flow at Kelso - Longview would be barely contained in the channel.

Therefore, with the present "existing conditions" at the SRS and in the Cowlitz River, all of the HEC - recommended flooding scenarios would be fully contained at Castle Rock and Kelso - Longview. The resulting flows would be similar to a 100 year flood event in the Cowlitz River. If the SRS were full of sediment, all of the HEC - recommended flooding scenarios would be fully contained at Kelso - Longview, but not at Castle Rock for either lake full or lake lowered 30 feet conditions. None of the hypothesized breaching and bulking scenarios will exceed or overtop the SRS for either "existing conditions" or "full conditions." It is expected, however, that for any major flooding event with the magnitude of the hypothesized breaching of Castle Lake, that significant quantities of sediment and debris will enter the SRS, thus reducing its present storage capacity and active life.

During large flood events the primary concern is usually for channel capacity and whether the peak discharge will be contained within the existing channel. The routing results presented in figures 10 through 13 and Table 5 show the beneficial effects of the SRS in reducing the peak discharges in the channels downstream from the SRS. Table 6 presents the average channel velocities computed at key locations below Castle Lake for the breaching and bulking scenarios that were considered. Maximum velocities occur just below the breach. For the HEC recommended scenario (with a bulking factor of 3.32) the maximum velocity just downstream from the breach is approximately 27 feet per second. Velocities of this magnitude occurring over the loose debris avalanche deposits can readily lead to the kinds of flow bulking scenarios described in Section 2.5. By the time the dambreak bore reaches the N-1 structure, it has decreased its velocity by almost half to 14.6 fps. This supports the debulking concept also presented in Section 2.5. Downstream from the N-1, the flow continues to slow down, but only slightly until it enters

Table 5

Routing Results at Key Locations Below Castle Lake for the HEC and USGS Breaching Scenarios

Run No.	Initial SRS Condition	Initial Castle Lake Elevation (NGVD)	Q_p Just Below Castle Lake (cfs)	Q_p at RM 5.3[1] (cfs)	Q_p at N-1 Structure RM 10.2[1] (cfs)	Inflow to SRS RM 16[1] (cfs)	Outflow from SRS Spillway (cfs)	Q_p at Castle Rock RM 48.0[1] (cfs)	Q_p at Longview Kelso RM 59.8[1] (cfs)
			Peak Flows, Q_p (cfs) for HEC's Dam Breach (Piping) Scenario, with BF=2.50[2]						
1	Existing	2,580 (full)	1,321,300	1,360,400	876,900	351,400	47,600	99,800	82,000
2	Existing	2,550 (-30')	900,500	746,400	523,400	186,500	18,800	77,100	72,100
3	Existing	2,520 (-60')	528,900	350,900	232,700	87,100	6,000	70,200	70,200
4	Full	2,580 (full)	1,321,300	1,360,400	876,900	351,400	108,900	143,000	104,600
5	Full	2,550 (-30')	900,500	746,400	523,400	186,500	79,500	122,300	93,500
6	Full	2,520 (-60')	528,900	350,900	232,700	87,100	40,600	96,900	81,900
			Peak Flows, Q_p (cfs) for HEC's Dam Breach (Piping) Scenario, with BF=3.32[3]						
7	Existing	2,580 (full)	1,321,300	1,540,000	1,136,000	695,000	105,200	139,700	109,400
8	Existing	2,550 (-30')	900,500	990,100	713,100	352,500	62,000	113,700	98,500
9	Existing	2,520 (-60')	528,900	461,400	340,500	131,800	6,000	70,200	70,200
10	Full	2,580 (full)	1,321,300	1,540,000	1,136,000	695,000	167,200	178,700	128,800
11	Full	2,550 (-30')	900,500	990,100	713,100	352,500	119,900	151,800	115,800
12	Full	2,520 (-60')	528,900	461,400	340,500	131,800	57,600	107,900	88,400
			Peak Flows, Q_p (cfs) for the USGS's Heave Scenario, with BF=4.46[4]						
13	Existing	2,580 (full)	1,692,000	2,311,400	1,827,900	1,024,300	184,800	189,900	134,000
14	Existing	2,550 (-30')	1,181,800	1,642,400	1,113,700	708,600	119,600	155,000	113,700
15	Full	2,580 (full)	1,692,000	2,311,400	1,827,900	1,024,300	237,000	230,300	154,800
16	Full	2,550 (-30')	1,181,800	1,642,400	1,113,700	708,600	179,100	192,300	135,100

[1] River Mile (RM) locations designate distance downstream from Castle Lake in miles.
[2] HEC's Bulking Scenario: BF = 2.50; n = 0.25, %Sat = 65%, Max Bulk Conc = 50% by Vol
[3] HEC's Bulking Scenario: BF = 3.32; n = 0.38, %Sat = 65%, Max Bulk Conc = 50% by Vol
[4] USGS's Bulking Scenario: BF = 4.46; n = 0.38, %Sat = 90%, Max Bulk Conc = 50% by Vol

Table 6

Computed Channel Velocities at Key Locations Below Castle Lake for the HEC and USGS Breaching Scenarios

Run No.	Initial SRS Condition	Initial Castle Lake Elevation (NGVD)	V_p Just Below Castle Lake (fps)	V_p at RM 5.3[1] (fps)	V_p at N-1 Structure RM 10.2[1] (fps)	Vel at Inflow to SRS RM 16[1] (fps)	Vel at Outflow from SRS (fps)	V_p at Castle Rock RM 48.0[1] (fps)	V_p at Longview Kelso RM 59.8[1] (fps)
			Max Vels, V_p (fps) for HEC's Dam Breach (Piping) Scenario, with BF=2.50[2]						
1	Existing	2,580 (full)	26.13	20.36	12.99	9.30	7.24	4.67	4.44
2	Existing	2,550 (-30')	22.56	17.13	10.52	6.97	4.98	3.98	3.70
3	Existing	2,520 (-60')	18.05	13.87	7.72	4.98	3.13	3.74	3.64
4	Full	2,580 (full)	26.13	20.36	12.99	9.30	9.72	5.68	5.30
5	Full	2,550 (-30')	22.56	17.13	10.52	6.97	8.74	5.20	4.89
6	Full	2,520 (-60')	18.05	13.87	7.72	4.98	6.81	4.55	4.43
			Max Vels, V_p (fps) for HEC's Dam Breach (Piping) Scenario, with BF=3.32[3]						
7	Existing	2,580 (full)	26.64	21.65	14.62	12.28	9.61	5.60	5.39
8	Existing	2,550 (-30')	22.61	18.72	12.13	9.31	7.99	4.95	4.43
9	Existing	2,520 (-60')	18.05	15.01	8.83	5.94	3.13	3.74	3.64
10	Full	2,580 (full)	26.64	21.65	14.62	12.28	11.14	6.52	6.09
11	Full	2,550 (-30')	22.61	18.72	12.13	9.31	10.03	5.88	5.64
12	Full	2,520 (-60')	18.05	15.01	8.83	5.94	7.78	4.83	4.68
			Max Vels, V_p (fps) for the USGS's Heave Scenario, with BF=4.46[4]						
13	Existing	2,580 (full)	28.63	23.61	16.80	13.93	11.48	6.73	6.27
14	Existing	2,550 (-30')	24.91	22.12	14.62	12.37	10.01	5.96	5.62
15	Full	2,580 (full)	28.63	23.61	16.80	13.93	12.38	7.42	6.94
16	Full	2,550 (-30')	24.91	22.12	14.62	12.37	11.37	6.73	6.33

1 River Mile (RM) locations designate distance downstream from Castle Lake in miles.
2 HEC's Bulking Scenario: BF = 2.50; n = 0.25, %Sat = 65%, Max Bulk Conc = 50% by Vol
3 HEC's Bulking Scenario: BF = 3.32; n = 0.38, %Sat = 65%, Max Bulk Conc = 50% by Vol
4 USGS's Bulking Scenario: BF = 4.46; n = 0.38, %Sat = 90%, Max Bulk Conc = 50% by Vol

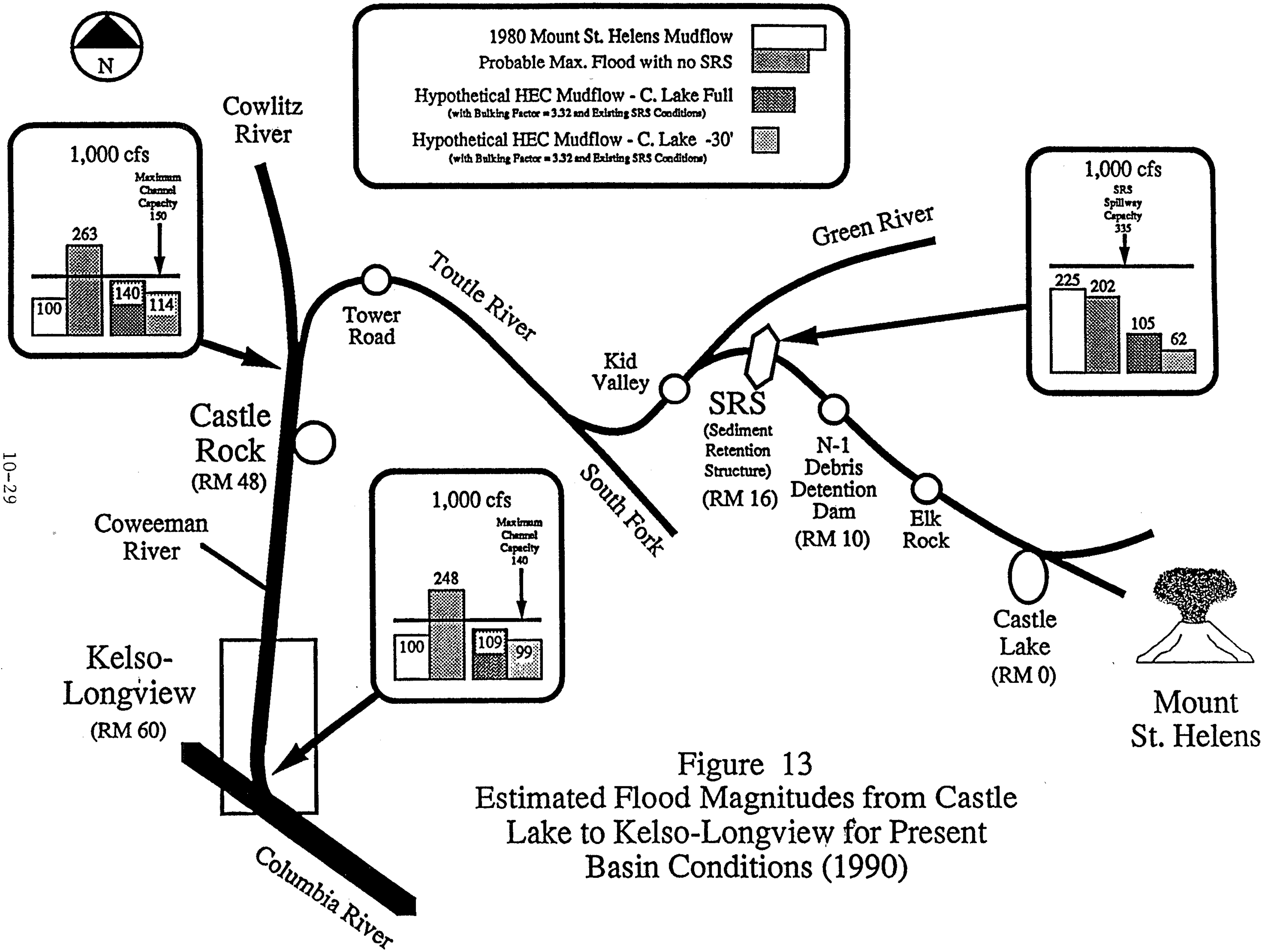


Figure 13
Estimated Flood Magnitudes from Castle Lake to Kelso-Longview for Present Basin Conditions (1990)

the SRS storage pool area (velocity here is approximately 12 fps). Flow leaving the SRS is controlled by the spillway and outlet works. The maximum average channel velocity just downstream from the SRS is approximately 9.6 fps. By the time the flood wave reaches the Cowlitz River and mixes with the cumulative base flow in the Cowlitz (69,800 cfs), it slows to about 4 to 6 feet per second.

The SRS protects communities and the river sections downstream from it but does not affect those areas upstream from the SRS. Therefore, there is no protection above the SRS from a hypothetical failure of Castle Lake. Because of this, and the possibility that a failure of Castle Lake would greatly reduce the sediment storage capacity and effective life of the SRS, additional field investigations pertaining to the geotechnical stability of the Castle Lake blockage and regularly scheduled monitoring are recommended.

4. Conclusions

The following conclusions are drawn from the results of this investigation:

1) Numerical methods developed by the National Weather Service (Fread, 1989), the Hydrologic Engineering Center, and Schaefer (1990), were successfully used to simulate the hypothetical breaching, flow bulking and unsteady mudflow routing that would result if the Castle Lake blockage dam were to fail.

2) Even though debris blockage dams form in a wide variety of physiographic settings, most debris blockage dams are very short lived. Costa and Schuster (1986) report that for the 63 documented cases they studied, 22 percent of the landslide dams failed in less than 1 day after formation and that half failed within a period of 10 days. Less than 10 percent of the natural debris blockage dams last more than 1 year.

3) More than 50 percent of the documented debris and landslide dams failed due to overtopping. The occurrence of a particular dam failure and the magnitude of resulting floods are predicated by: the size of the blockage; its geometric characteristics (size and depth of the impoundment, and size and shape of the blockage); the properties of the blockage materials; the rate of filling of the impoundment; the volume of the trapped water; and bedrock or engineered controls such as spillways, tunnels and diversions.

4) The Castle Lake blockage was ten years old in May, 1990 and appears to be stable under its past and present conditions. The Portland District Corps of Engineers installed an emergency spillway in October of 1981 to stabilize the lake elevation at 2577 feet NGVD. Groundwater levels in the blockage and seeps along the downstream face of the blockage have been monitored since the eruption. According to the Corps' Geotechnical Branch (personal communication, 1990), they have seen no field evidence of unstable conditions in the blockage materials since the installation of the spillway.

5) The estimated peak discharge from a hypothetical failure of the Castle Lake blockage exceeds the peak discharges predicted from potential energy versus peak discharge relationships developed from historical dam failures by more than 2.3 times (see Figure 7). It exceeds the predicted peak discharge envelope curve from historical dam failures by 3.6 times (see Figure 8). Therefore, the Corps recommended breaching and bulking scenario produces a conservative estimate (i.e., flows that are larger than those observed during historical failures of similar blockages) of the possible peak discharge that could result during a breaching of the Castle Lake Blockage.

6) The effect of the SRS is significant in reducing the peak discharge from the hypothetical failure of Castle Lake. The initial elevation of the lake prior to failure also affects the magnitude of the resulting dambreak discharge. For the failure and bulking scenario

recommended by the Corps of Engineers, the SRS reduces the peak discharge into the North Fork Toutle River by 85 percent (from 695,000 to 105,200 cfs) for full lake conditions. If Castle Lake is lowered 30 feet prior to its failure, the SRS reduces the peak flow by 82 percent (from 352,500 to 62,000 cfs). If Castle Lake is lowered by 60 feet, the SRS reduces the peak flow by 95 percent (from 131,800 to 6,000 cfs).

7) The amount of storage the SRS can provide depends on how full of sediment it is when a flood event occurs. Under the present "existing conditions" in the SRS and the Cowlitz River, all of the Corps recommended flooding scenarios would be fully contained at Castle Rock and Kelso - Longview. The resulting flows would be similar to a 100 year flood event in the Cowlitz River. If the SRS were full of sediment, all of the Corps recommended flooding scenarios would be fully contained within the channel at Kelso - Longview, but not at Castle Rock for either lake full or lake lowered 30 feet conditions.

8) None of the hypothesized breaching and bulking scenarios will exceed or overtop the SRS for either "existing conditions" or "full conditions."

9) The SRS protects communities and those river sections downstream from it, but does not affect the areas upstream from the SRS. Therefore, there is no protection above the SRS from a hypothetical failure of Castle Lake. Because of this, and the possibility that a failure of Castle Lake would greatly reduce the sediment storage capacity and effective life of the SRS, additional field investigations pertaining to the geotechnical stability of the Castle Lake blockage and continuous monitoring are recommended.

5. References

Costa, John E. (1984) "Physical Geomorphology of Debris Flows," in Developments and Applications of Geomorphology, by Costa, J.E. and Fleisher, P.J., eds., Berlin, Springer-Verlag, pp. 269-317. Society of America, Inc., Boulder, CO.

Costa, John E. and Schuster, Robert L. (1986). "A Perspective on Landslide Dams," in Landslide Dams: Processes, Risk, and Mitigation, by Schuster, Robert L., American Society of Civil Engineers, New York, New York.

Costa, John E. and Schuster, Robert L. (1988). "The Formation and Failure of Natural Dams," in Geological Society of America Bulletin, Vol. 100, pp. 1054-1068, July 1988.

Fairchild, Lee H. (1987). "The Importance of Lahar Initiation Processes," in Debris Flows/Avalanches: Process, Recognition, and Mitigation, by Costa, John E. and Wieczorek, Gerald F., eds., The Geological

Fread, D.L. (1989) "BREACH: An Erosion Model for Earthen Dam Failures," Users Manual, National Weather Service, Hydrologic Research Laboratory, Silver Spring, Maryland.

Fread, D.L. (1989) "DAMBRK: The NWS Dam-Break Flood Forecasting Model," Users Manual, National Weather Service, Office of Hydrology, Silver Spring, Maryland.

Froelich, D.C. (1987). "Embankment-Dam Breach Parameters," in Proceedings of the 1987 National Conference on Hydraulic Engineering at Williamsburg, VA, American Society of Civil Engineers, New York, New York.

Laenen, Antonius and Orzol, L. L. (1987). "Flood Hazards Along the Toutle and Cowlitz Rivers, Washington, from a Hypothetical Failure of Castle Lake Blockage," Water-Resources Investigations Report 87-4055, U.S. Geological Survey, Portland, OR.

MacArthur, Robert C. and Hamilton, Douglas L. (1989). "Numerical Simulations of Mudflows Induced by Hypothetical Failure of the Castle Lake Blockage near Mount St. Helens, Washington," Projects Report No. 89-04, Hydrological Engineering Center, Davis, CA.

MacDonald, T.C. and Langridge-Monopolis, J. (1984). "Breaching Characteristics of Dam Failures," in Journal of Hydraulic Engineering, American Society of Civil Engineers, Vol. 110 (5), pp. 567-586.

Meyer, D.F. and Dodge, J.E. (1988). "Post-Eruption Changes in Channel Geometry of Streams in the Toutle River Drainage Basin, 1983-85, Mount St. Helens, Washington," Open-File Report 87-549, U.S. Geological Survey, Vancouver, WA.

Schaefer, M.G. (1990). Personal communication and hand written notes. Dam Safety Section, Water Recources Program, Department of Ecology, Olympia, WA 98504-8711.

U.S. Army Corps of Engineers, Portland District (1990). "Unpublished Data: Soil Material Properties for the Area Below Castle Lake to the N-1 Structure," collected by the Geotechnical Branch of the Portland District, July 1990, Portland OR.

U.S. Army Corps of Engineers, Portland District (1988). "Castle Lake: Engineering Analysis and Alternative Evaluation," Portland OR.

U.S. Army Corps of Engineers, Portland District (1984). "Mt. St. Helens Cowlitz and Toutle Rivers Sedimentation Study," Portland, OR.

U.S. Army Corps of Engineers, Hydrologic Engineering Center (1985). "Toutle River Mudflow Investigation," Special Projects Report No. 85-3, Davis, CA.

U.S. Department of the Interior, Bureau of Reclamation (1973). Design of Small Dams, a Water Resources Technical Publication, Washington, D.C.

Appendix A

Derivation of the equation to calculate the "Ultimate Bulking Factor"

By

Melvin G. Schaefer, Ph.D., P.E.

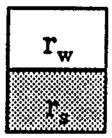
Dam Safety Section
Water Resources Program
Department of Ecology
Olympia, WA 98504-8711

MUDFLOW BULKING

Definitions:

r_w = ratio of water to unit volume in mudflow.
r_s = ratio of soil to unit volume in mudflow.

Mudflow



V_w = volume of initial free water
V_s = volume of soil in "mudflow"
Vol = volume of ultimate mudflow

<u>If source soil is totally dry</u>:

$$Vol = \overline{V_w} + \overline{V_s}$$

where:

$$\overline{V_s} = r_s\ Vol$$

$$\overline{V_w} = r_w\ Vol$$

$$Vol = \frac{\overline{V_w}}{r_w}$$

$$\overline{V_s} = \frac{r_s}{r_w}\overline{V_w}$$

then,

$$Vol = \overline{V_w} + \frac{r_s}{r_w}\overline{V_w}$$

$$Vol = V_w(1 + \frac{r_s}{r_w})$$

If source soil available has water in void:

$$\begin{array}{|c|l} \hline V_w & \text{- water} \\ \hline V_s & \text{- soil} \\ \hline \end{array} \Big\} \text{in-situ soil mass}$$

where given unit of soil mass has

$$\frac{V_w}{V_s} = \theta, \qquad V_w = \theta V_s$$

$$\textit{then,} \quad Vol = \overline{V_w} + \frac{r_s}{r_w}\overline{V_w} + ((\frac{r_s}{r_w})\,\overline{V_w})\,\theta + ((\frac{r_s}{r_w})\,\overline{V_w}\,\theta)\,(\frac{r_s}{r_w}) +$$

$$((\frac{r_s}{r_w})^2\,\overline{V_w}\theta)\theta + ((\frac{r_s}{r_w})^2\,\overline{V_w}\theta^2)(\frac{r_s}{r_w}) + \ldots\ldots$$

define

$$\frac{r_s}{r_w} = \phi$$

or

$$\phi = (\frac{r_s}{1-r_s}) \textit{ by volume}$$

then

$$Vol = \overline{V_w}(1 + \phi + \phi\theta + \phi^2\theta + \phi^2\theta^2 + \phi^3\theta^2 + \phi^3\theta^3 + \quad + \ldots)$$

or

$$\begin{aligned} Vol = \overline{V_w} &+ \overline{V_w}(\phi\theta + \phi^2\theta^2 + \phi^3\theta^3 + \ldots\ldots) \\ &+ \overline{V_w}(\phi + \phi^2\theta + \phi^3\theta^2 + \ldots) \end{aligned}$$

for

$$Vol = \overline{V_w} + \overline{V_w} \sum_{n=1}^{\infty} (\phi\theta)^n \qquad + \overline{V_w} \sum_{n=1}^{\infty} \phi^n \theta^{n-1}$$

$$\phi\theta \geq 1 \rightarrow \textit{series limit} \rightarrow \infty$$

then

$$Vol = \overline{V_w} + \overline{V_w} \sum_{n=1}^{\infty} (\phi\theta)^n - \overline{V_w} + \frac{\overline{V_w}}{\theta} \sum_{n=1}^{\infty} (\phi\theta)^n - \frac{\overline{V_w}}{\theta}$$

$$Vol = \left(\frac{\overline{V_w}}{1-\phi\theta}\right) + \frac{\overline{V_w}}{\theta}\left(\frac{1}{1-\phi\theta}\right) - \frac{\overline{V_w}}{\theta}$$

$$Vol = \left(\frac{\overline{V_w}}{1-\phi\theta}\right) + \left(\frac{\overline{V_w}\phi}{1-\phi\theta}\right)$$

$$Vol = \overline{V_w}\left(\frac{1 + \phi}{1 - \phi\theta}\right)$$

Define:

in mudflow by volume:

$$\phi = \left(\frac{r_s}{r_w}\right) = \left(\frac{r_s}{1-r_s}\right)$$

in-situ soil mass (source material):

$$\theta = \left(\frac{V_w}{V_s}\right)$$

Then:

$$\boxed{\text{Bulking Factor}} \rightarrow \left(\frac{1 + \phi}{1 - \phi\theta}\right)$$

% solids	ϕ	θ						
		0	.10	.20	.30	.40	.50	.60
20%	.25	1.25	1.28	1.32	1.35	1.39	1.43	1.47
30%	.43	1.43	1.45	1.56	1.64	1.73	1.82	1.93
40%	.67	1.67	1.79	1.93	2.09	2.28	2.51	2.79
50%	1.00	2.00	2.22	2.50	2.86	3.33	4.00	5.00
60%	1.50	2.50	2.94	3.57	4.55	6.25	10.0	25.0
70%	2.33	3.33	4.34	6.24	11.1	49.0	00	00
80%	4.00	5.00	8.33	25.0	00	00	00	00

(by volume)

These Bulking Factors are "ultimate values" which assume the physical factors, such as: slope, discharge, turbulence and entraining mechanisms, channel hydraulic properties, soil source, gradation, sediment properties, etc; are operating for sufficient time to allow this value to be attained.

Example #1

Assumptions

1) Ultimate mudflow is 50% solids by volume (r_s = .50)

$$\phi = \left(\frac{r_s}{r_w}\right) = \frac{r_s}{1-r_s}$$

$$\textit{For}\ \ 50\%\ \left(\frac{V_s}{Vol}\right): \qquad r_s = 0.50\ \ ,\ \ \phi = 1.00$$

and,

2) Porosity,

$$\textit{Porosity}\ (n) = 0.28 = \left(\frac{e}{1+e}\right)$$
$$\textit{(geometric property)}$$

where,

$$e = >.39$$
$$\textit{(void ratio)}$$

for mudflow case → assume 90% of voids are filled with water available to make up mudflow

then, effective void ratio:

$$\hat{e} = .90(.39) = .35$$

and,

$$\theta = \hat{e} = \left(\frac{V_w}{V_s}\right) = .35$$

Potential Bulking Factor for 50% solids by volume mudflow with ϕ = 1.00 and θ = 0.35

Bulking Factor =

$$\left(\frac{1+\phi}{1-\phi\theta}\right) = 3.08$$

Ultimate Mudflow =

19000(3.08) = 58,520 acre-ft (castle lake)

Example #2

For ultimate mudflow with 55% solids by volume (r_s)

Porosity (n) = .30

95% saturation of voids

then,

$$\phi = \frac{r_s}{1-r_s} = 1.22$$

and,

$$n = .30 = \frac{e}{1+e},\ e = .43$$

$$\theta = \frac{V_w}{V_s}\ \textit{source material}$$

$$\theta = .95\ (.43) = .41$$

Potential Bulking Factor =

$$\textit{Bulking Factor} \quad B.F. = \left(\frac{1+\phi}{1-\theta\phi}\right) = 4.44$$

Ultimate Mudflow =

19000 acre-ft (4.44) = 84,360 acre-ft

Obviously, expected values and upper and lower bounds for bulking can be computed in this manner for selected values of r_s, ϕ and θ

Abstract

Atlantic Salmon Restoration in New England

by Townsend Barker
New England Division
US Army Corps of Engineers

In pre-colonial times, at least 20 major rivers in New England contained significant Atlantic salmon populations. The number of adult salmon entering these rivers annually may have exceeded 300,000 individuals. These runs were brought to extinction through overharvesting and dam building in five of the six New England States (Massachusetts, Connecticut, Rhode Island, New Hampshire, and Vermont) by the mid-nineteenth century, and were reduced to remnant populations in Maine by the early part of this century. Moderately successful efforts to restore salmon in some rivers in the late 1800's demonstrated it was possible to construct functional fish passage facilities at existing dams. However, overfishing was not controlled and water quality declined through this period into the first half of the twentieth century making salmon restoration impractical. Now with improved water quality and controls on fishing, there is an ongoing cooperative effort by federal and state agencies to restore self-sustaining populations of Atlantic salmon by the year 2021 to much of the species' historical range in New England.

One of the biggest rivers in New England and one with large nursery areas potentially available to adult salmon is the Connecticut, and one of its tributaries with excellent salmon habitat is the West River. However, two Corps of Engineers flood control dams, Ball Mountain and Townshend Lakes, are obstacles to upstream and downstream migration on the West River. This paper describes approaches being used to restore Atlantic salmon to New England in general, and involvement of the Corps in constructing fish passage facilities on the West River in particular.

ENVIRONMENT Magazine, Vol. 31, No. 10,
pp. 12-20, 29-31, December 1989

Overcoming Federal Water Policies

The Wildcat-San Pablo Creeks Case

By Ann L. Riley

The average time spent planning a U.S. government-assisted flood-control project before construction begins is 26.1 years.[1] These delays are a direct result of federal policies and practices that conflict with some basic community needs. The deficiencies in federal water-project planning policies and their impacts on U.S. communities are manifest in the 33-year history of a flood-control project in North Richmond, California. North Richmond is an impoverished, unincorporated community in Contra Costa County on the eastern shore of San Pablo Bay, a northern extension of San Francisco Bay (see the map in Figure 1 on page 15).

North Richmond grew up during World War II when blacks who came to work in the shipbuilding industry were segregated on the floodplains of Wildcat and San Pablo creeks. The creeks flood and cause poor drainage in the vicinity almost every winter, but more severe flooding puts North Richmond under a foot of water about once every three years.[2] The community's need for flood control has never been disputed. However, the problems inherent to federal policies regarding the design and funding of flood-control projects have repeatedly delayed its implementation. During that time, the community has initiated herculean efforts and innovations to overcome federal obstacles to funding such projects for poor communities; designing projects that recognize local goals for economic recovery and environmental quality; and adjusting to the technical vulnerabilities of traditional flood-control channelization.

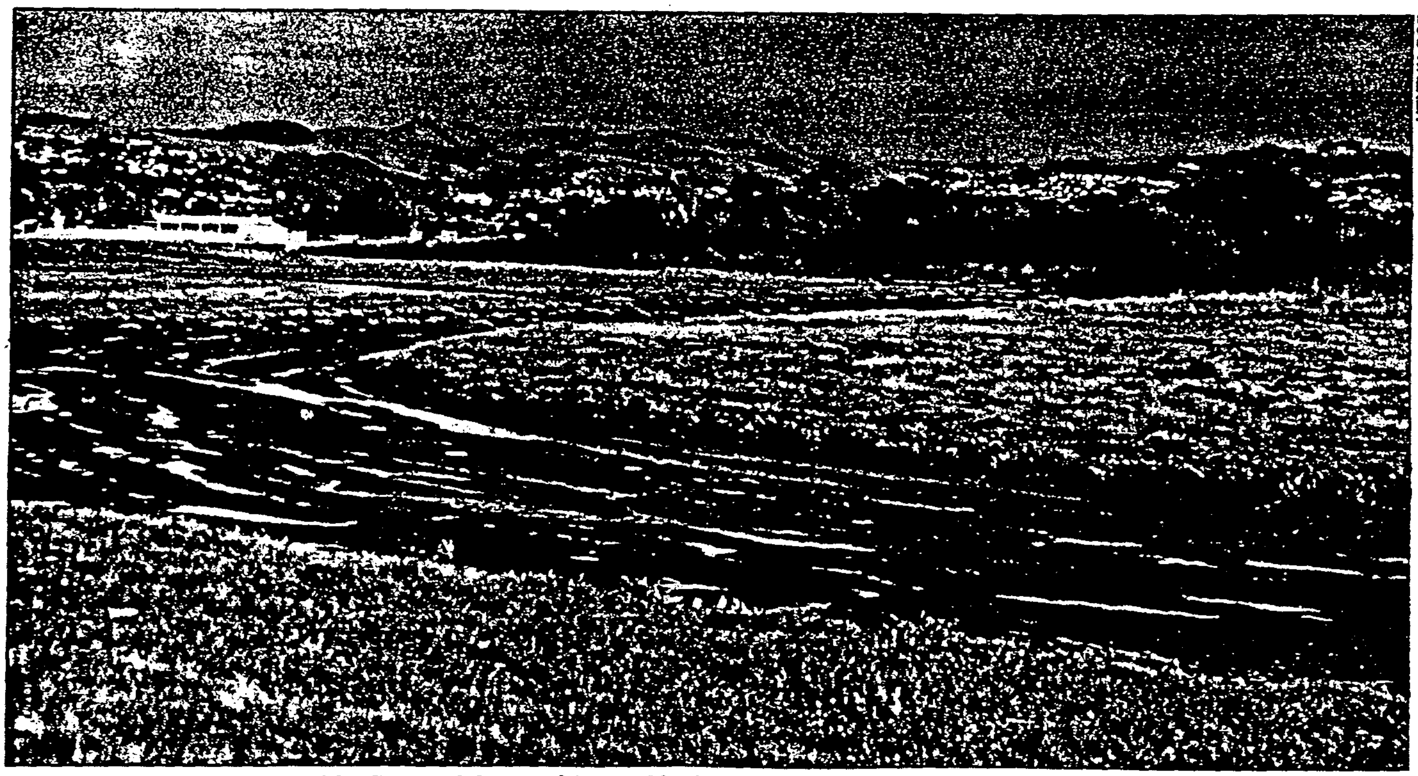
North Richmond is considered by the U.S. Department of Housing and Urban Development (HUD) to be one of the most impoverished communities in the country and, therefore, deserv-

ANN L. RILEY is on leave from her position as chief of the Financial Assistance and Environmental Review Branch of the California Department of Water Resources in Sacramento, California. Her involvement in the flood-control project for Wildcat and San Pablo creeks has been as a citizen volunteer and not as a government representative.

ing of federal assistance. (The 1980 census classified 64.5 percent of the households in North Richmond as female-headed and below the poverty level.) However, suburban development in other parts of Contra Costa County has made the county as a whole one of the wealthiest in California. Economic redevelopment and improvement in the standard of living in North Richmond are unlikely to be achieved without a flood-control project. Although the community has atypical demographics because it is mostly composed of minorities, the residents' values and goals reflect those of other communities: They want opportunities, options, and environmental quality, and they want to have influence in the decisions that affect them. If North Richmond's need for flood control has been met only with the greatest difficulty by the federal water-project planning process, then something is wrong with federal policies and practices.

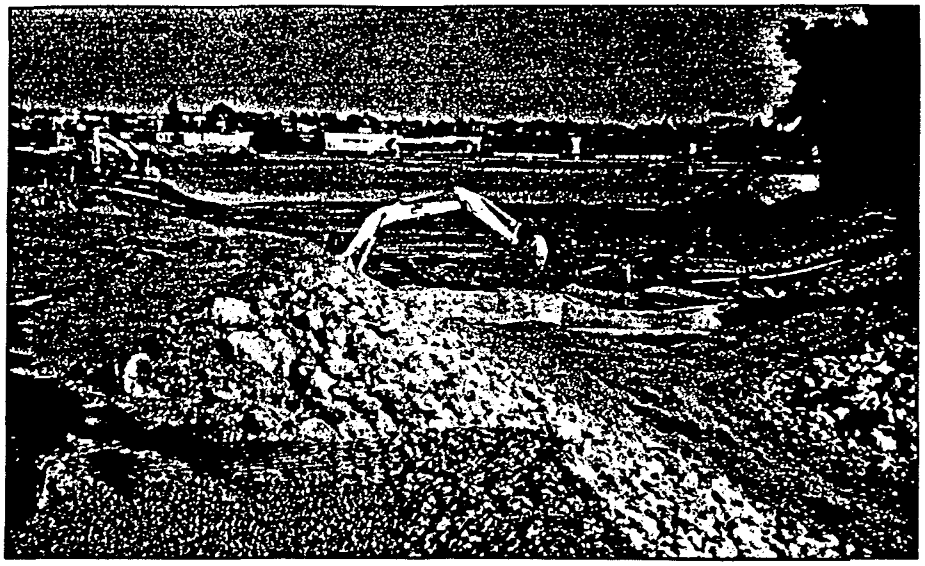
Early Efforts

In the 1940s and early 1950s, flooding along the Wildcat and San Pablo creeks attracted attention to North Richmond's need for flood control. By 1956, the Contra Costa County Flood Control District had assessed that need and issued a report calling for the implementation of a flood-control project. As a result, in the 1960 Flood Control Act, Congress authorized the U.S. Army Corps of Engineers to conduct a feasibility study for flood control on the two creeks. At that time, the standard practice for reducing flood damages was to construct costly and environmentally damaging reservoirs and stream channels that carry more water at a higher velocity than could be carried by the natural channels. However, national experts in geography, hydrology, engineering, and economics were recommending that the federal government broaden its approach to the re-



BOB WALKER

The flora and fauna of San Pablo Creek marsh were threatened by a flood-control plan of the U.S. Army Corps of Engineers.



Bulldozers dig a basin to trap sediment from Wildcat Creek. Without the trap, sedimentation would harm the marshland habitat downstream. (Photo: Bob Walker)

duction of flood damages.[3] The experts recommended greater use of nonstructural means of reducing damages, such as floodplain zoning, flood proofing, and relocation of structures, and suggested that a wider range of project sizes be considered. They also recommended that the design of projects be based on more complete data on the watershed and on broader social, environmental, and economic objectives. In 1962, the Harvard Water Program published *Design of Water-Resource Systems*,[4] which presented the recommendations of the best available expertise on how to improve federal water-project planning policy. One of the document's most important recommendations was to base planning on multiple objectives, such as economic growth, regional income distribution, and environmental quality, rather than on the construction of single-purpose engineering works.[5]

In 1968, the Army Corps of Engineers issued a report that presented several different flood-control plans, but no plan was recommended for implementation because the foreseen benefits of the project did not pass the federal cost-benefit test. The only benefits the federal government recognizes in a cost-benefit analysis are tied to the values of the structures in the flood-hazard area that would receive protection. In North Richmond, the substandard housing—some of it just cardboard boxes—was not valuable enough to justify a project.

Multi-Objective Planning in the 1970s

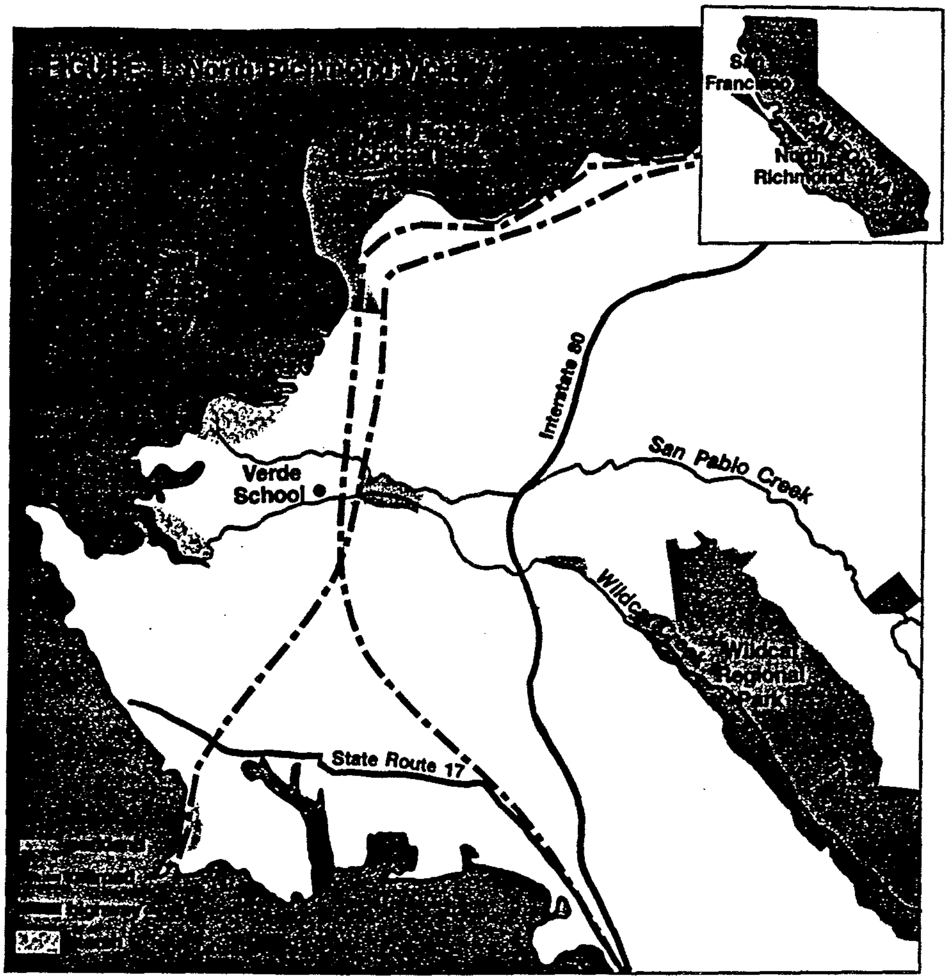
The National Environmental Policy Act of 1969 required the federal government to establish a process for the public review of the impacts of federal projects. (For more details on this law, see Lynton K. Caldwell's article beginning on page 6 of this issue.) In 1974, a new Water Resources Development Act required the consideration of nonstructural alternatives in flood-control planning, and revisions to the federal Water Resources Council's principles and standards made between 1973 and 1979 integrated environmental and social objectives into the cost-benefit analysis of proposed water projects.

Earlier, however, HUD had started the Model Cities Program for urban renewal, and, by 1971, a plan for Richmond was developed that featured Wildcat and San Pablo creeks and the San Pablo Bay shoreline as a recreational and commercial resource to serve as a focus for the redevelopment of the area (see Figure 2 on page 16).[6] The Richmond Model Cities Plan called for HUD to take flood control off the shelf, and HUD proceeded to contract for a privately prepared economic analysis of a flood-control project.[7] Eleven years after the first federal studies began, political momentum succeeded in overcoming the difficulty of the cost-benefit analysis; HUD's consultants considered future project benefits and potential recreational benefits and made the numbers work.

With new, favorable cost-benefit formulas from HUD's consultants, the corps of engineers conducted a planning process that reflected the pressures of the 1970s to increase public participation in project planning and produced a new, community-supported flood-control plan that was authorized by Congress in 1976. A case study written on this phase of the Wildcat-San Pablo flood-control project, *Can Organizations Change?*, praised the corps' first effort to accommodate the needs of a poverty-stricken area.[8] The corps based its planning on the multiple objectives of the Richmond Model Cities Plan, which focused on social well-being, environmental quality, and economic redevelopment. The project benefits included protection of existing and future development, the expected increase in market value of the project area, and recreational benefits. North Richmond residents involved in the project planning during this era were complimentary of the corps' planning process and sensitivity to community needs.[9]

The corps of engineers considered including an Environmental Quality Plan among its project alternatives. Although they did not choose the Environmental Quality Plan as the Recommended Plan, neither did they choose the National Economic Development (NED) Plan, which was a single-objective plan to reduce flood damages. The NED Plan maximized the difference between costs and benefits for a project designed to provide protection against the 100-year flood (that is, a flood of such magnitude that it is likely to occur only once in 100 years). The Recommended Plan adopted by the corps in 1979 contained traditional flood-control engineering for the 100-year flood in the form of concrete box culverts and trapezoidal and rectangular concrete channels, but the plan also provided for a dirt, trapezoidal channel on lower Wildcat Creek that would have some landscaping. Also authorized as part of the flood-control project were several recreational elements, including a regional trail, a nature study area near Verde Elementary School (which stands beside Wildcat Creek), and freshwater impoundments on ponds.[10]

Federal policy requires that all land acquisitions, easements, right-of-ways, and up to 50 percent of the recreation components be paid for by the community. When North Richmond set about raising its share of the expense for this project, some of the area's major businesses—including Chevron Oil; Southern Pacific Railroad; Atchison, Topeka and Santa Fe Railroad (which had a train derail over San Pablo Creek in a January 1982 storm); and the Richmond Sanitary Company—did not contribute. Their parsimony contributed to the community's failure to raise the required local share of the total cost. Thus, the federal cost-sharing requirements undermined the corps' efforts to design a plan that would use the creeks as part of a community economic revival plan, as outlined in the Richmond Cities Plan.



Under the Reagan Administration

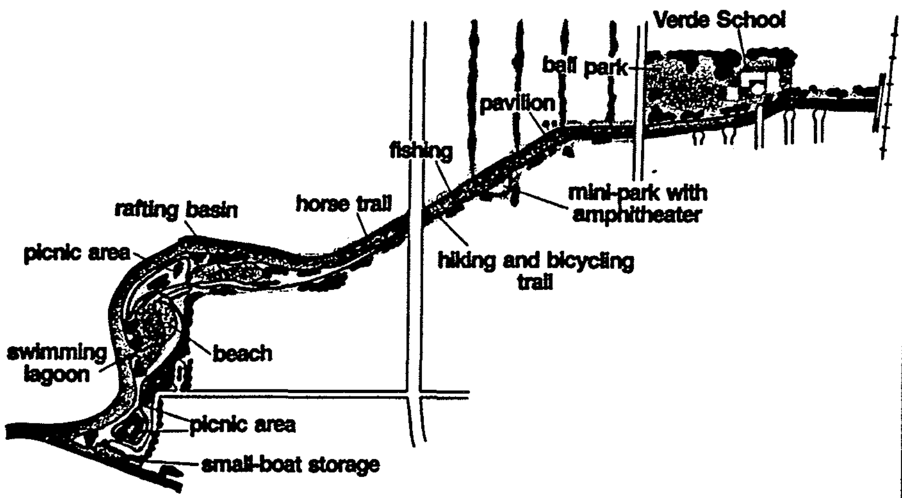
In the 1980s, federal policies reverted to favoring the construction of projects based on a single objective of economic efficiency. The Reagan administration's standards and guidelines required the selection of a NED Plan that was described by the corps' staff as a least-cost plan to reduce flood damages; neither environmental quality nor nonstructural plans were supposed to be considered in the development of project alternatives. The administration also required local residents to pay a greater portion of the project costs in addition to the cost of land acquisition, easements, and right-of-ways.

In 1982, Contra Costa County officials proposed a bare-bones, structural flood-control project without any environmental amenities to be constructed in cooperation with the Army Corps of Engineers. The county board of supervisors, as the local sponsor, presented the "Selected Plan" to the North Richmond community on a take-it-or-leave-it basis and argued that it was the only affordable alternative (see Figure 3(a) on page 19). Although the corps' staff demonstrated more openness by being willing to discuss alternative plans with the public, the corps decided to take a back-seat role and defer to the county on the issues of project design and citizen participation. The corps of engineers also discouraged multi-objective planning in the belief that North Richmond could not afford anything but a basic channelization project.

Some North Richmond residents were resigned to accepting any flood-control project offered; others felt so strongly about the Richmond Model Cities Plan that they wanted to retain influence in the design process and explore other project options. The take-it-or-leave-it option ran counter to the long history of active community involvement in the Richmond Model Cities Plan and alienated some key community leaders. In the spring of 1983, community leaders organized a meeting in North Richmond to determine community reaction to the county/corps Selected Plan for flood control. The issues raised at that meeting defined the next five years of work for the community volunteers who changed both the planning process, the plan design, and funding strategy.

Members of several North Richmond community groups, including the Richmond Neighborhoods Coordinating Council, the Urban Creeks Council, Save San Francisco Bay Asso-

FIGURE 2. Richmond Model Cities Plan for Wildcat Creek.



SOURCE: Joint Agency Committee for the Development of North Richmond–San Pablo Bay Area, "North Richmond–San Pablo Bay Area Study—Summary Report" (Contra Costa County, September 1971).

ciation, and the Contra Costa County Shoreline Parks Committee, formed a coalition to request that a plan be developed that recognized the value of Wildcat and San Pablo creeks as important local and regional resources and that recognized the regulatory, funding, and technical design problems inherent in the county's proposed plan.

The coalition raised several important environmental concerns:

• Wildcat Creek was classified by the California Department of Fish and Game as one of the last remaining streams in the San Francisco Bay area with almost a continuous riparian environment along its length. However, the county/corps Selected Plan would make it a concrete and earth-lined channel complete with covered box culverts.

• Environmental experts, including two nationally prominent hydrologists, Luna Leopold and Phil Williams, feared that the project would, through sedimentation, do serious harm to the wetlands and marshes of the lower floodplain. Hydrologists reported to the coalition that the corp's estimates of sediment moving through the two creeks were substantially too low; that the concrete-lined channels would not provide the flood protection assumed by the project's designers because the sediment would increase the hydraulic resistance and decrease the capacity of the channels; that the plan would create costly and frequent maintenance needs; and that the proposed sediment detention basin on Wildcat Creek would not protect the marshland of the lower floodplain from sedimentation.

• There were no sponsors or plans to provide recreational open space and educational benefits for members of the community and other regional park users.

Other issues associated with the Selected Plan were the safety hazards of locating a box culvert for high-velocity storm flows next to Verde Elementary School; obstacles to getting regulatory approval from state and federal agencies; and the difficulty of raising the local share of the plan's cost, given the Reagan administration's demand for increasing local cost-sharing requirements and the plan's unattractiveness to other potential federal and state funding contributors.

Despite the efforts of the Grizzly Peak Flyfishers, the East Bay Regional Park District, and the California Department of Fish and Game to increase public and political awareness of the environmental issues by planting native trout in Wildcat Creek in September 1983, the county remained opposed to broadening the project's objectives or responding to technical reviews. Therefore, the Urban Creeks Council and the Richmond Neighborhoods Coordinating Council decided to design their own flood-control plan and successfully applied to the charitable Vanguard Foundation in San Francisco and the San Francisco Foundation for funding. The coalition of neighborhood and environmental organizations used a 1960s organizing and community participation strategy known as advocacy planning, in which it solicited its own paid and unpaid experts to develop a new "Modified Plan" to compete with the county/corps Selected Plan.

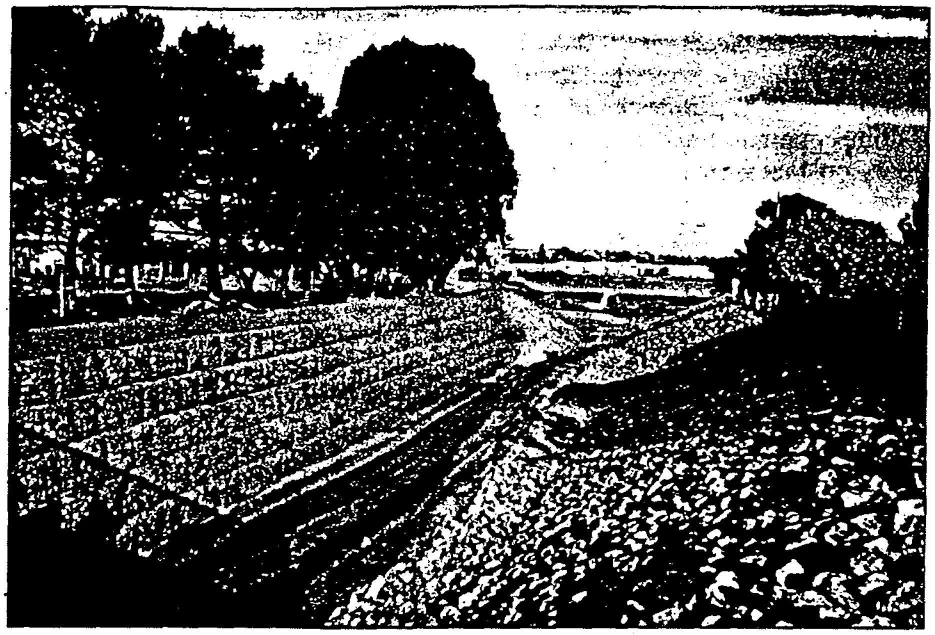
The Modified Plan

The East Bay Regional Park District was an early supporter for developing a plan that would allow for the extension of popular regional trails from Wildcat Canyon and Point Pinole Shoreline parks along Wildcat and San Pablo creeks and their marshes. Financial assistance from the park district and the Save San Francisco Bay Association brought the coalition's final, alternative planning budget to $50,000, enough to pay for the design of a flood-control project on at least one of the creeks, although the design's principles and many of the details would, of course, be applicable to both creeks. Eventually, a Modified Plan for Wildcat Creek was developed with a very different design philosophy from that of the Selected Plan.[11] This new plan would modify the existing creek channels to simulate the natural hydraulic shape and processes of undisturbed streams, deposit the sediment in the upstream floodplain, and restore valuable riparian vegetation. The proposed concrete and trapezoidal earth channels of the Selected Plan were replaced in the Modified Plan with more natural, low-flow, meandering channels, floodplains, setback levees, planted gabion walls, and riparian trees (see Figure 3(b) on page 19). The Modified Plan also included regional trails and park facilities. The coalition's planners developed their own project cost estimates and funding plan and presented their Modified Plan at all the same meetings attended by the public and government agencies at which the Selected Plan was presented.

The advocacy planning strategy introduced alternatives and, therefore, controversy into the Army Corps of Engineers' planning sessions. The strategy eventually forced a change in the planning process from one in which citizens were to be briefed on the final Selected Plan chosen by the county board of supervisors and the corps of engineers to one in which citizens became active participants in determining the design of the final plan. Also, citizen participation evolved from a Citizen Advisory Committee with handpicked members who could be depended on to vote for the Selected Plan to an open process in which anyone affected by the plan could help to determine the design.

With the county, corps of engineers, and community at loggerheads, the staff of state Assemblyman Bob Campbell helped to negotiate a planning process that used combined government-citizen design and funding teams to arrive at some consensus. Campbell's staff also helped North Richmond residents meet their share of the project costs by identifying state funding sources made accessible by the broader objectives of the final "Consensus Plan." Thus, the coalition used its Modified Plan to force the consideration of a multi-objective plan back into the planning process.

On 19 February 1985, the Contra Costa County Board of Supervisors approved the Selected Plan for construction but left the door open for multi-objective designs if funds became available. In June 1985, the U.S. Fish and Wildlife Service had reviewed the Selected Plan and issued their legally required Biological Opinion, which prevented the corps from implementing the Selected Plan because of its probable impacts on the marshes and their endangered species. The Fish and Wildlife Service then adopted the coalition's Modified Plan as "the prudent and reasonable alternative."[12] In addition, the San Francisco Bay Conservation and Development Commission did not find the Selected Plan consistent with the requirements of the McAteer-Petris Act for the protection of San Francisco Bay wetlands.[13] But the commission found it could permit the Modified Plan. A combination of pressure from federal and state environmental and regulating agencies, the endurance and persistence of community leaders, and press coverage resulted in the adoption by the Contra Costa County Board of Supervisors of a multi-objective Consensus Plan. Construction on the Consensus Plan began in 1987 and still continues.



Under the Consensus Plan, this part of Wildcat Creek is lined with gabions (on the left) and a rock bank. The trees have been saved and more native species will be planted. (Photo: Bob Walker)

Design by Consensus

When the corps of engineers found, in June 1985, that it could not implement the Selected Plan, the county board of supervisors established a project design team to construct a plan in which the concerns of the government agencies with regulatory powers over the project would be properly coordinated and integrated with the concerns of the public. The design team was not formed because an enlightened county or corps aimed to pioneer consensus planning; it was formed out of a crisis situation caused by the lack of support for the project on the part of state and federal regulatory agencies and by the negative publicity the proposed Selected Plan had generated. The team was to produce a fundable project that the regulatory agencies would accept and that the coalition could endorse. Team members included representatives from the U.S. Fish and Wildlife Service, the California State Lands Commission, the California Department of Fish and Game, the San Francisco Bay Conservation and Development Commission, the California Coastal Conservancy, the East Bay Regional Park District, state Assemblyman Bob Campbell's office, state Senator Dan Boatwright's office, Congressman George Miller's office, the coalition and its own professional experts, local land and nursery owners, and, of course, the Contra Costa County Flood Control District and the U.S. Army Corps of Engineers. Meetings occurred no less than once a month, and, in 1985, the meetings were sometimes scheduled as often as once a week. Throughout the planning effort of the next three years, attendance at the design team's meetings remained high, averaging approximately 20 persons per meeting.

Competition among the different interests on the team resulted in many grueling meetings. An important turning point in the consensus-making process was the appointment of Jim Cutler as chairman of the design team. Cutler, a neutral person from the county planning department with good group management skills, replaced the county engineer, who had a personal bias for a single-objective design. The other key component to the success of the consensus design process was that the county paid the citizen's own hydraulic expert, Phil Williams, who had helped design the Modified Plan, to represent the coalition at design team meetings. The ultimate measure of success of the consensus planning process was that, after an unsuccessful, 29-year planning history, the flood-control project was designed and funded and construction had begun within two years. Two notable problems arose: the first, when relevant and interested parties were not included on the design team; and the second, when continuity in decisionmaking and plan formulation broke down because of continual changes in corps and county staffing. The first problem occurred because the Richmond Unified School District Board was not adequately involved in the design of the project, which ran through their property near Verde Elementary School. The school board held up the project by withholding the right-of-way until its concerns were met. The school board also used the advocacy planning strategy by hiring a consultant to design an alternative plan. By withholding the right-of-way, the school board was able to force a more environmentally sensitive treatment of the part of the creek running through school property.

The other difficult problem that plagued the design team was the lack of continuity in both the federal and local staff assigned to the project. Between 1984 and 1988 the corps of engineers assigned three different engineers to the job of project manager. The resultant discontinuity in decisionmaking brought on an environmental and publicity disaster featured in a front-page article in the *San Francisco Examiner-Chronicle* on 14 June 1987.[14] Construction plans that did not reflect the decisions of the design team were given to the contractors who accordingly bulldozed a half mile of riparian vegetation that was supposed to be preserved. Shortly thereafter, a levee constructed in the wrong location prevented the implementation of a marsh restoration project and jeopardized state funds for the marsh enhancement plan. The situation was exacerbated when a key member of the county staff gave the construction contractors approval to proceed with plans that did not correspond to the team's decisions. To prevent further problems, the design team adopted a new system of taking team-approved minutes in addition to publishing and mailing cross-sections and maps of the approved stream channel and project designs to all design team members.

Design Features

The design team chose features for the Consensus Plan from the designs of the Modified and Selected plans already proposed. Although the design team's final Consensus Plan is a compromise between the two plans, the basic components of the Modified Plan were retained because of the importance of managing the large amount of sediment, particularly in the Wildcat watershed, to avoid degrading the endangered species' habitat in the marshes (see Figure 3(c) on page 19).

One of the most important features of the coalition's Modified Plan was that the stream corridors, or floodways, would remain within the same narrow right-of-way boundaries that the 1982 county's Selected Plan used and would provide the same level of protection against a 100-year flood. The right-of-ways of the corps' original 1976 plan had been up to 250 feet wide to accommodate certain environmental features. The Modified Plan, however, included riparian vegetation next to the channels and a terrace for sediment accumulation but did not increase the project's width beyond 180 feet. Yet the designs of the Modified Plan that were incorporated into the Consensus Plan provided the same level of flood protection as the 1976 design because a different design philosophy was used in which the channels were modeled not on the dimensions or performance of a hydraulic flume but on natural channel geometry. Thus, the design of the Consensus Plan disproves the common presumption that only trapezoidal or rectangular channel geometry can be used in a narrow project right-of-way.

Ultimately, sections of the right-of-ways in the Consensus Plan were increased because state and local entities purchased or donated lands to enhance the project. For example, the State Lands Commission purchased some downstream land on Wildcat Creek between the riparian area and the marsh to provide a transition zone that would enhance the environment and catch sediment. Upstream on Wildcat Creek, the school district donated additional land for the right-of-way to provide more and better design options. The county had never presented these options to the school board. Because of design problems with the sediment basin, corps and county officials concluded that the basin should be relocated to an upstream site. This change ultimately raised the land acquisition costs for the project.

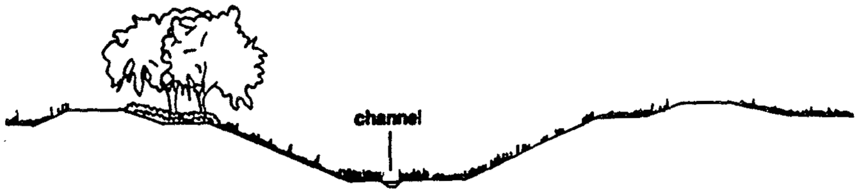
The Consensus Plan substituted the standard trapezoidal dirt and riprap channels, rectangular concrete channels, and box culverts of the Selected Plan with natural floodplain features of the Modified Plan wherever possible. The Consensus Plan has 10- to 15-foot-wide, meandering, low-flow channels designed to carry the creek's 1.5 recurrence interval flows (mean flows) and floodplains where the flows could spread, lose velocity, and deposit sediment. Riparian vegetation is included on both sides of the low-flow channels and riparian trees will shade the channels and prevent the growth of bulrushes and willows, which obstruct flow. Although previous corps project designs had designated a low-flow channel in lower Wildcat Creek, they did not include natural channel geometry or vegetation or grading plans that would help define stable, low-flow channels. Typically, the corps' low-flow channels superimposed on open, wide-bottom, trapezoidal channels are unstable, braided, and choked with bulrushes.

The Consensus Plan is designed so that sediment deposition will occur where it is least harmful—on the floodplain and in the bay. By trapping as much sediment in the upstream floodplains as possible, filling of the downstream marsh with sediment should be prevented. The Consensus Plan assures that the low-flow channels will scour and transport as much sediment as possible to San Pablo Bay. To further protect the marsh from sedimentation, the plan also calls for widening the slough channels through the marsh so that suspended sediments can be conveyed by the channels without overtopping into the marsh and for excavating sediment to increase the brackish marsh area and restore the marsh's tidal action.

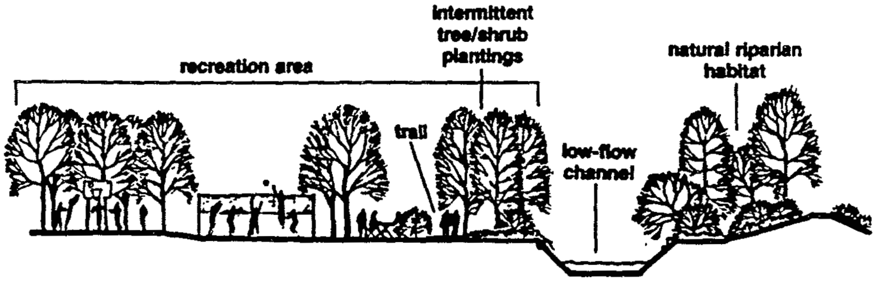
Technical Issues

The most contentious technical issues faced by the design team included making reasonable estimates of the sediment loads carried by the creeks, assessing the ability of the corps' proposed sediment basin to collect sediment, judging the safety of concrete box culverts, and assigning roughness values to proposed revegetation areas. The coalition's experts argued that the natural creek channels were aggrading with high sediment loads and predicted that the even wider, trapezoidal channels proposed by the corps would further increase sedimentation. The narrow, low-flow channels of the Modified Plan, therefore, were better designed to transport sediment in suspension at higher velocities. Phil Williams and Luna Leopold also questioned the ability of the corps' proposed sediment basin to perform as a sediment trap. Later the corps' own specialists at the Waterways Experiment Station in Vicksburg, Mississippi, independently raised the same concern. Therefore, the design team decided to locate the basin further upstream, and they adopted the floodplains, wetland transition zone, and higher velocity, low-flow channels of the Modified Plan to keep the sedi-

FIGURE 3. Cross-sections of creek channels designed for the Selected, Modified, and Consensus plans for the flood-control project on Wildcat and San Pablo Creeks in North Richmond, California.



(a) 1982 Selected Plan proposed by Contra Costa County and U.S. Army Corps of Engineers.



(b) 1984 Modified Plan proposed by a coalition of North Richmond community organizations.

(c) 1986 Consensus Plan developed by a design team of community, county, and federal representatives.

SOURCES: U.S. Army Corps of Engineers, *General Design Memorandum and Basis of Design for Reach 1, Wildcat and San Pablo Creeks* (Sacramento, Calif.: U.S. ACE, Sacramento District, October 1985); Poster of the Modified Plan published by a coalition of North Richmond community organizations including the East Bay Regional Park District; and U.S. Army Corps of Engineers, *Supplement No. 3 to Design Memorandum 1, Wildcat and San Pablo Creeks Environmental Mitigation Project* (Sacramento, Calif.: U.S. ACE, Sacramento District, August 1988).

ment load from ending up in the marsh or significantly decreasing the channels' capacity.

Another difficult design issue to resolve was how to make up for the loss of 24 acres of riparian vegetation. The county's 1982 proposal called for planting trees on some acreage north of Wildcat Creek. In the Consensus Plan, trees planted along the two creeks' low-flow channels would help guide channel formation and shade the bank to prevent it from clogging with rushes, reeds, and sediment. However, county engineers did not want vegetation near the channels because they felt this would make channel maintenance difficult for them. Thus, choosing roughness values that would determine how much vegetation could be allowed without reducing the needed channel capacity became a critical aspect in the design of the Consensus Plan.

Roughness values are calculated by using the Manning Equation to describe the flow resistance caused by the texture of the surface over which the water must flow. But the assignment of roughness values is a very subjective process. The corps originally considered using the values 0.100 for the riparian areas south of the low-flow channels and 0.045 for the north floodplains. (Lower roughness values mean more vegetation is allowable.) The design team finally decided that a composite value for the low-flow channels and south bank riparian forests would be 0.050 (conditional upon maintaining clear low-flow channels), and a roughness value of 0.035 was assigned to the north bank floodplains for low shrubs and grasses.

Once roughness values had been chosen, the design team had to agree upon a maintenance plan for keeping the low-flow channels cleared of vegetation until a riparian canopy could grow to shade out the unwanted, clogging reed growth expected in exposed, low-flow channels. The agreement negotiated between the county supervisor and the corps' project manager provides for inexpensive hand labor by conservation crews to clear the unwanted vegetation. Potential maintenance crews include the State of California Conservation Corps and a local East Bay Conservation Corps as well as labor from the state's new workforce program. It was also agreed that the standard, annual maintenance routines for removing sediment or clearing vegetation would be substituted with a maintenance schedule based on actual need. Thus, maintenance activities, costs, and negative environmental impacts resulting from channel maintenance should be reduced.

Maintenance

The consensus maintenance plan is one of the most important innovations of this project. Federal government policy mandates that local project sponsors must accept long-term responsibility for the maintenance of any project. But corps officials readily admit that such maintenance costs have been grossly underestimated over the years. These costs may have been underestimated simply because they fall on the costs side of the cost-benefit analyses, but another likely reason for the misjudgment is that the corps' channelization projects have not performed as the engineers expected. Many flood-control channels quickly re-establish their original grades when sediment fills in the project's designed grade, thus greatly reducing the channel capacities. Lowered capacity results in more frequent and more expensive maintenance bills.

Because the design team also had to face the reality of the project's limited maintenance budget, a critical need of the Consensus Plan was to provide a channel design that would reflect the equilibrium in a natural system and that would assume a certain amount of sediment deposition in the calculation of channel capacities. The Wildcat-San Pablo Creek Maintenance Master Plan was as much a negotiated part of the design team's Consensus Plan as the project features. It requires an annual field inspection of the project by interested agencies and community organizations. The Hydraulic Engineering Center-2 water surface profile model will be used to estimate channel capacity at cross sections selected for monitoring. When vegetative growth and sediment deposition reduce the two creeks' freeboards by 50 percent, participants in the maintenance planning will prescribe how to thin the vegetation and/or remove sediment to re-establish the channels' capacity while minimizing maintenance activity impacts on the environment.

To design a revegetation plan that would reflect the needs of the U.S. Fish and Wildlife Service, the California State Lands Commission, and other members of the design team, the county asked the corps of engineers to contract with the Soil Conservation Service, which has experience with the revegetation and restoration of streams. In September 1988, the Soil Conservation Service and the corps issued a recreation and revegetation supplement to the corps' design memorandum about the Consensus Plan.[15] Their revegetation design objective is not to landscape a flood-control project but to restore a riparian environment along the low-flow channels. Revegetation will be done with cuttings from nearby plants, seeds from California species native to the locale, and some container stock. Because of the competence demonstrated by the landscape architects in the design process, the design team asked the corps to retain the Soil Conservation Service staff for the actual plant installation.

The most significant test of this innovative project remains, however: to complete construction according to the design team's plans and specifications. The Army Corps of Engineers estimates that construction should be completed in 1990.

The Funding Strategy

The coalition's Modified Plan and the county's Selected Plan had very similar cost estimates. The Consensus Plan's costs were higher because the sediment basin was redesigned and relocated. The transition of this project from a single-objective flood-control

(continued on page 29)

Federal Water Policies

(continued from page 20)

project to a multi-objective project to restore marshes, provide recreational and educational opportunities, and enhance the environment, as well as to control flood damages, made it possible to attract funding from state agencies that could not otherwise have contributed. For example:

- The East Bay Regional Park District committed $793,000, which was matched by another $793,000 by the corps, for a regional trail system. The park district later committed $19,000 to help enhance creekside educational opportunities near Verde Elementary School and may commit more as the recreational and educational project element is finalized.
- The California State Lands Commission purchased $240,000 worth of land for the Wildcat Creek wetland transition zone.
- In February 1987, the California Coastal Conservancy Board authorized the expenditure of $578,000 for marsh restoration and riparian enhancement areas. After the original restoration plan was damaged by the construction mistakes in the Wildcat and San Pablo creek marshes and the county failed to identify willing sellers of riparian land parcels, the Coastal Conservancy headed a task force to come up with a new marsh restoration plan. A total of $46,000 was used from the first Coastal Conservancy authorization, $5,000 was provided to the design team effort, and a second authorization of $314,870 was committed by the conservancy's board to implement a revised restoration plan.
- In June 1989, the California Department of Water Resources awarded a $100,000 grant because the project involved design innovations, a commitment to citizen participation, and educational opportunities.

As of fall 1989, the Consensus Plan has attracted funds totaling more than $2 million. A project finance committee composed of local, state, and federal representatives and agency staffs has not yet completed its fund-raising activities, and there are reasonable chances of more state, park district, or foundation monies becoming available.

The federal project cost-sharing policy in the 1980s has been to increase nonfederal contributions for projects and to use a community's willingness and ability to pay as an important criterion for selecting projects for construction. Such policies discriminate against a poor community trying to meet its share of the large costs associated with a water project. The most effective strategy for helping North Richmond to raise its share of the cost was to diversify the project and attract state dollars. Unfortunately, this strategy resulted in a difficult Catch-22. By attracting more dollars to the project for these diverse benefits, which are actually classified as project costs by federal standards, the costs side of the cost-benefit ratio was raised and might have upset the required ratio between costs and benefits for project approval. The corps' project manager cleverly adapted to this impossible situation by classifying the marsh restoration, some of the riparian areas, the wetland transition zone, the Verde Elementary School revegetation and educational area, and the park district staging area as enhancements occurring outside the project's boundaries and, therefore, not part of the official project costs. Because the corps classified these project components as enhancements they became the financial responsibility of the community. Ultimately, the transition zone was made a project requirement by the Endangered Species Act as a control point for sediment catchment and had to be included in the project costs equation.

Federal policies for project evaluation and funding are strongly biased against a project like North Richmond's flood control. Federal definitions of water-project costs and benefits do not reflect the broad, long-term needs and values of the communities where such projects are often located. Likewise, the federal cost-sharing policies do not recognize unique, local economic and social conditions. The policies discriminate against financially disadvantaged communities attempting to benefit from federal projects even though these communities are frequently located in some of the most hazardous areas. Because the cost-sharing policies make it a local responsibility to purchase lands, easements, and right-of-ways, there is a built-in bias against the purchase of riparian preservation zones, trails, and other environmental features.

Policies and Practices

At the same time as corps officials, Congressman George Miller, and local representatives were brandishing their shovels at the project's ground-breaking ceremony in October 1986, a new policy for the authorization and design of water projects was being set out by the 1986 Water Resources Development Act. After a follow-up water omnibus bill was passed in 1988, the corps issued the *Digest of Water Resources Policies and Authorities*, which is used as a policy guide for the development of water projects by corps personnel.[16] There are no provisions in these policies to design environmental quality plans or nonstructural alternatives as part of the flood-control planning process. The main component of the new policy is to increase substantially the nonfederal share of project costs. Accommodating the financial need of a community is left to the discretion of the assistant secretary of the army in charge of civil works. The corps is to build NED plans that maximize net benefits, and any project enhancements beyond this are to be paid for solely by the community. This policy translates into the assumption that the corps will construct channelization projects for flood control, but that environmental features of some kind can be tacked on only if the community pays for them. The new policy also maintains the barrier against any model in which a different design philosophy is used to build more natural, stable channels integrated with other environmental features.

A teacher takes his students to explore Wildcat Creek. The creek, which runs along the south side of Verde Elementary School, presents many educational opportunities. (Photo: Alan La Pointe)

There are some possibilities for improving the policies and practices outlined in the corps' digest. For example, the policies have left open the possibility that communities may select smaller projects than what is needed for protection from the 100-year flood. This kind of choice is based on the rationale that, if the locals are going to pay for more of the project, they should be able to have more say in the project design.

Even though North Richmond is a federally recognized poverty area, the assistant secretary of the army in charge of civil works did not respond to the request of Congressman Miller to provide a larger federal share of the project cost. This refusal may be credited to North Richmond's location in an affluent county. Revenues for flood-control projects are raised by assessing the districts where the projects are located. But in coastal California, it is not unusual for poorer communities to be located in downstream floodplains while the wealthy live on the upstream hills where no flood hazards exist. Typically, segments of the population who live adjacent to projects but do not benefit from them do not elect to fund the projects. Federal cost-sharing policies and the assistant secretary of the army need to be more realistic about local socioeconomic conditions. If North Richmond, with a median annual income of $7,412 and a 64.5 percent poverty rate, cannot qualify for flexible cost-sharing arrangements, then what community will?

In the interest of holding down federal water-project expenditures, the federal government clings to the use of an outmoded cost-benefit analysis and an inequitable cost-sharing system that are biased against low-income areas and nonstructural solutions. Even the environmental lobby supports the federal cost-sharing policies in the belief that such policies will reduce the number of projects and thus reduce damage to the environment. The endorsement of such policies strikes a blow to rational planning in which plans are designed to fulfill desirable objectives. It is inconsistent and contradictory for environmental advocates to challenge the use of the cost-benefit analysis as an oversimplified means to justify the selection of projects for federal assistance but to accept the use of cost-sharing arrangements as a critical aspect of the project justification process. Moreover, the cost-benefit analysis and the cost-sharing system should not be the only determinants for qualifying projects for federal support; local priorities, needs, and objectives must be incorporated into the plans, as should broader national goals for social and environmental needs.

Federal water-project planning has been and will continue to be driven on the basis of the scarce federal dollar. The great irony of the impasse is that a reformed system using objectives-based planning and technical designs based on concepts of hydrology instead of channel hydraulics would reduce both the federal share of costs and the total project construction bill. Objectives-based planning will save federal dollars because:

- the projects that will legitimately meet the test of fulfilling multiple objectives are few;
- different technologies, such as stream restoration strategies, can lower project costs;
- different construction and maintenance techniques may contribute to local economies just as the Works Progress Administration did in the 1930s and 1940s; and
- protection measures against the smaller, more frequent floods instead of the larger, 100-year floods will reduce the cost of many projects.

Citizen participation is considered by many water-project planners to be a costly nuisance, but many project engineers and members of Congress can tell of dramatic planning-cost overruns that occurred after years of studies and planning when citizens blocked projects after they were authorized or before construction started. Most federal water-project planners do not realize that a high level of citizen participation can attract financial contributors to projects. Citizen participation can also stimulate political support and interest in a project, and such support is crucial to attracting project money from a diversity of local, county, regional, and state programs. In addition, just as the multiple objectives of the Consensus Plan brought in nonfederal funds, projects that meet more than one objective, such as park development, fisheries enhancement, recreation, and wildlife benefits, save federal dollars by attracting other funding sources, such as state and local resource, fish-and-game, and park agencies.

Some nonstructural and environmentally sensitive design measures do incur higher land acquisition costs. But these costs need to be balanced against the long-term costs of maintaining structural engineering works, constant sediment removal, vegetation removal, and the unintended impacts common to the traditional project design. Fiscally responsible policymaking and project design must weigh the true, long-term costs of traditionally designed projects against the costs of land acquisition.

The U.S. Army Corps of Engineers is proud of the flood-control project on Wildcat and San Pablo creeks. An engineer for the Sacramento district wrote an article for *Hydraulic Engineering* describing the interesting hydraulics of the Consensus Plan.[17] The corps' Waterways Experiment Station has encouraged the use of this project as a model for future water-project designs in training courses. However, well-intentioned corps personnel who want to respond to local needs in formulating plans find themselves caught between conflicting local needs and federal policies. Over the last 10 years, the project in North Richmond is just 1 of 12 California water projects that the public has tried to redesign to meet community needs.

The current federal system of water-project evaluation is so narrow that only those communities with the most influential representatives will be able to circumvent the planning system through a long and costly process and get a project that meets community needs. Such a system does not stop pork-barrel projects; it only makes them more time-consuming and expensive. Only a system that recognizes the need for multi-objective planning and ensures that these objectives are met by the project under consideration for federal assistance will produce water development projects with genuine local and national benefits.

NOTES

1. Government Accounting Office, "Update on Army Corps of Engineers' Planning and Designing Time for Water Resources Projects," GAO/RCED-84-16 (Washington, D.C.: U.S. Government Accounting Office, January 1984).

2. U.S. Army Corps of Engineers, *General Design Memorandum and Basis of Design for Reach 1, Wildcat and San Pablo Creeks* (Sacramento, Calif.: U.S. ACE, Sacramento District, October 1985).

3. These early experts included Gilbert White, Jim Goodard, Otto Eckstein, John Krutilla, William Hoyt, Walter Langbein, Luna Leopold, Thomas Maddock, Arthur Maass, Maynard Hufschmidt, and many others.

4. Arthur Maass et al., *Design of Water-Resource Systems, New Techniques for Relating Economic Objectives, Engineering Analysis, and Government Planning* (Cambridge, Mass.: Harvard University Press, 1962).

5. Martin Reuss, "Interview with Professor Arthur Maass," 20 May 1983, Harvard University.

6. Joint Agency Committee for the Development of North Richmond-San Pablo Bay Area, *North Richmond-San Pablo Bay Area Study Summary Report* (Contra Costa County, Calif., September 1971).

7. INTASA, "Relationship of Proposed Flood Control Project and Model Cities Objectives for Community Development in North Richmond" (Menlo Park, Calif., June 1971).

8 Daniel A. Mazmanian and Jeanne Nienaber, *Can Organizations Change?* (Washington, D.C.: The Brookings Institution, 1979).

9. Barbara Vincent, Jay Vincent, Lucretia Edwards, Laura Hunter, Jean Siri, and members of U.S. Army Corps of Engineers planning committees, conversations with author, 1983.

10. U.S. Army Corps of Engineers, *Wildcat-San Pablo Creeks, Contra Costa County, California, Feasibility Report for Water Resources Development* (San Francisco, Calif.: U.S. ACE, August 1973; U.S. Army Corps of Engineers, *Wildcat-San Pablo Creeks Contra Costa County, California, General Design Memorandum Phase I for Flood Control and Allied Purposes; Draft* (San Francisco, Calif.-U.S. ACE, December 1977); and U.S. Army Corps of Engineers, *Master Plan Wildcat and San Pablo Creeks* (San Francisco, Calif.: U.S. ACE, Plan prepared by Arbegast, Newton & Griffith Landscape Architects, Contra Costa County, Calif., 1979, Draft).

11. Philip Williams and William Vandivere, *A Flood Control Design Plan for Wildcat and San Pablo Creeks* (Sponsored by the San Francisco Foundation, the Vanguard Foundation, the East Bay Regional Parks District, and Save San Francisco Bay Association, 27 February 1985).

12. U.S. Fish and Wildlife Service, "Endangered Species Formal Consultation on the Proposed San Pablo and Wildcat Creek Flood Control Project, Contra Costa County, California," AFA-SE 1-1-85-F-19 (Portland, Oreg.: U.S. Fish and Wildlife Service, 12 June 1985).

13. McAteer-Petris Act of 1965 was the State of California's enabling legislation that authorized the formation of the San Francisco Bay Conservation and Development Commission. The act requires the regulation of fill placed in the bay and requires public access to the bay.

14. Eric Brazil, "Trouble Anew for 'Murdered' Creek," *San Francisco Examiner-Chronicle*, 14 June 1987, A-1.

15. U.S. Army Corps of Engineers, note 2 above.

16. U.S. Army Corps of Engineers, *Digest of Water Resources Policies and Authorities*, EP1165-2-1 (Washington, D.C.: U.S. ACE, 15 February 1989).

17. Edward F. Sing, "Stable and Environmental Channel Design," *Hydraulic Engineering*, August 1988.

Mud Mountain Dam, Flood Control Challenges In a Hostile Environment

James L. Lencioni, P.E. [1]

Introduction

Mud Mountain Dam was constructed by the Seattle District during the 1940's to control flooding along the Puyallup River downstream from the city of Puyallup in western Washington state. The 425-ft high, rolled-filled earth embankment is actually located on the White River, a major tributary to the Puyallup River, and approximately 30 miles upstream from the authorized flood control area. Mud Mountain Dam controls an approximately 400 square mile portion of the White River's drainage basin. At the time of construction, the White River downstream from the dam was largely undeveloped.

The White River drains the northern and northeastern slopes of Mount Rainier originating from the Emmons and Winthrop Glaciers. The stream gradient between the sources and the head of the reservoir averages about 100 ft per mile (0.018 ft/ft). Mount Rainier, with a peak elevation of 14,411 ft, has experienced numerous volcanic eruptions over it's history and dominates the geologic and sediment characteristics of the White River basin. The river's sediment inflow to the Mud Mountain Dam reservoir is very high, and varies in composition between fine sand and silt carried during the summer month's glacier melt period and large sand, gravel and cobbles during the winter rain-generated flood events.

Mud Mountain Dam is operated as a single purpose flood control storage project. The reservoir is kept essentially empty except during periods of high inflow during the winter months when water is stored behind the dam to control flooding downstream. Water is then released as fast as possible after the peak of the inflow to provide storage for subsequent floods. The cycle of storing and releasing water is repeated as often as storm conditions require. During a typical flood season, 2 to 3 flood events having durations on the order of one week generally occur.

The existing flood control hydraulic features consist of two separate tunnels serviced by individual intake towers, both of which would be deeply submerged during large flood events. One tunnel is of a 9-ft horseshoe-shape with a discharge capacity of approximately 5,200 cfs. The other tunnel is a 23-ft diameter concrete-lined structure which trifurcates into three, 8.5 ft diameter steel pressure pipes having a total capacity of approximately 12,400 cfs. Modifications to the intake towers and

[1] Hydraulic Engineer, U.S. Army Corps of Engineers, Seattle

outlets works are presently under construction to ensure flood control and reservoir drawdown capability during and following major flood events. Critical design considerations included identification of the amount and spatial depositional pattern of sediments inflowing to the reservoir during major flood events, intake tower location and subsequent tunnel alinement constraints necessary to economically interface with the existing structures, and the approximately 300-ft design head on the tunnels.

Dam-Safety Assurance Study

Dam-safety assurance studies in the mid-1980's identified several dam safety deficiencies at the existing project. The issues directly related to the flood control works included potential debris and sediment blockage of the intake towers and/or tunnels during large floods similar to those expected under project design to spillway design conditions. Such blockage conditions would significantly impact flood control and after-flood drawdown capability of the project with subsequent impact upon other geotechnical concerns at the project. Another concern with the original design included the recurring damage and periodic maintenance repair required in the 9-ft tunnel every 2-3 years as a result of the large amounts of large-sized sediments carried through that tunnel.

Reservoir Sedimentation Investigations

General. Determination of an estimate of the amount of deposition near the immediate vicinity of the flood control intake tower was of paramount concern in design of an intake tower to provide necessary flood control and drawdown hydraulic capacity in view of the large sediment loads expected. The project is operated essentially as a run-of-the river reservoir except during flood events when water is stored for flood control at Puyallup and then released relatively quickly following the inflow event. Such an operation is conducive to minimizing long-term sediment deposition in the reservoir, therefore the dam-safety sedimentation investigation was limited to short-term flood events. In addition, the design criteria was limited to water-flood events only, i.e., mudflow-type conditions which could exist with volcanic eruption of Mount Rainier were considered too remote and beyond the capability of effective defensive design.

Determination of Sediment Load. The only sediment inflow data available consisted of two partial years of suspended sediment measurements made by the United States Geological Survey (USGS) in 1975 and 1976. The maximum river discharge at which these measurements were made was approximately 15,000 cfs. River discharges up to about 250,000 cfs are associated with the spillway design flood at Mud Mountain.

Extrapolation of the USGS measured sediment concentrations to the larger flood discharges considered in design modifications to the outlet works resulted in projected sediment concentrations as high as approximately 650,000 milligrams per liter (mg/l). Concentrations of this magnitude would exceed those projected for the North Fork Toutle River based on measurements taken in 1984 (3 years after the basin-devastating eruption of Mount St. Helens). Such concentrations are representative of hyperconcentrated and mud flow sediment transport regimes not considered reasonable for water-flood events on the White River.

Investigations using observed sediment concentrations and yields from the December 1964 floods on the Mad and Eel River basins in northwest California eventually formed the basis for developing a design condition sediment load vs water discharge relationship for the Mud Mountain outlet works design. These basins are morphologically similar to the White River basin. Sediment sources during large hydrological events on the White River would be from mass wasting, channel bed, bank, and overbank scour; the same types of sediment-producing processes dominant on the Mad River in the 1964 event. The unit discharges associated with the sediment measurements made on the Mad River during the 1964 flood were 2 to 3 times higher than those at which the USGS had collected sediment data on the White River. The sediment inflow rating curve ultimately adopted for design considerations was a curve representing a best-fit regression of the measured White River data extrapolated to a discharge of 23,600 cfs at which point the slope of the curve changes to follow the best-fit regression for the 1964 data measured on the Mad River. Using this relationship, the maximum sediment concentration at the peak discharge of the Mud Mountain spillway design flood is predicted to be 182,000 mg/l.

Sediment Distribution in Reservoir. Estimating the depth of sediment deposition in the near vicinity of the flood control intake tower was necessary in order to size the intake tower. A relatively simplistic sediment budget/reservoir detention period analysis was used to estimate sediment deposition conditions for which the intake tower geometry and configuration was designed to accommodate. Reservoir trap efficiency was based upon published data from the Garrison Reservoir and the sediment deposit topset slope was assumed to be 50 percent of the natural stream slope based on guidance from various published reports. The procedure used reasonably simulated depositional conditions which had previously occurred during the 1977 flood event at Mud Mountain. Using this procedure, sediment deposition depths of about 90 ft and 150 ft were predicted with the project design and spillway design flood, respectively.

In order to address reviewer concerns regarding the potential lack of conservatism associated with the sediment budget analysis, an HEC-6 (NETWORK version) sediment transport numerical model analysis of reservoir sediment transport and deposition was accomplished. Recognizing that HEC-6 is best suited to evaluating long term bed profile response to hydrologic and sediment input rather than single event simulation, this analysis was used primarily to provide a "qualitative check" of the results of the sediment budget analysis.

Sediment transport predictions for the White River are complicated by the high concentrations of suspended sand and silts, and the large sized gravel and cobbles in the channel bed. The transport function used in the analysis was Madden's 1985 modification of the Laursen function. Steady-state simulations at river discharges less than about 50,000 cfs indicated that the river's transport capacity was greater than the previously discussed sediment inflow curve. This resulted in HEC-6 model predictions of extensive scour in the upstream reaches of the reservoir. Review of reservoir sedimentation range survey data revealed that such scour conditions were not evident. The sediment load curve for the HEC-6 simulations was therefore adjusted so that sediment inflow approximated upstream river transport capacity as computed by HEC-6. Once adjusted, the HEC-6 model closely simulated the approximately 40-50 ft of deposition which occurred at the intake tower during the 1977 flood event.

Design Condition Simulations. The adjusted HEC-6 model was run for two large flood conditions, i.e., the project design and spillway design floods. The simulations predicted the expected sedimentation processes of deposition at the head of the reservoir during the rising limb of the inflow hydrograph when water was being stored behind the dam, and subsequent resuspension and deposition of sediment further downstream in the reservoir during the pool drawdown after the flood event. The model ultimately predicted deposition depths of approximately 70 and 90 ft near the dam embankment with the project design and spillway design floods, respectively, and a deposit topset slope of about 0.006 (about 60 percent of the natural streambed slope). The results of the HEC-6 simulations were considered to confirm the reasonableness of the previously developed intake tower design parameters.

Modifications to Flood Control Outlet Works

Criteria. Design modifications to the flood control outlet works were developed based on the following hydraulic-related criteria:

a. The modified design must meet the originally authorized project design flood discharge and reservoir capability and must minimize, to the extent possible, reservoir drawdown time with the probable maximum flood event when considering sediment and debris conditions.

b. Sedimentation conditions would be based upon water-flood events only. Mudflow-type conditions which could exist from volcanic eruption of Mount Rainier were considered too remote and beyond the capability of effective defensive design.

c. The modified design must provide upstream control for the 23-foot diameter tunnel and include provisions for emergency closure for both tunnels.

Hydraulic Design. Numerous concepts were investigated to develop the most cost-effective structural modification. The hydraulic design constraints included (a) sizing of the intake tower entrance area to ensure required discharge capacity when considering sediment and debris deposition, (b) providing an acceptable tower location and tunnel alignment and transition to acceptably tie into the existing tunnels, (c) ensuring that flood control capability was maintained throughout the construction and diversion period, (d) consideration of the high velocities (up to 120 fps at design pool elevation) existing in the 23-foot tunnel entrances and the horizontal curve tieing in the new and existing portions of the 23-foot diameter tunnel, and (e) design of an acceptable exit structure for the 23-foot diameter tunnel.

Intake Tower. A single intake tower servicing the entrances to both tunnels was considered to be the most effective design concept. A tower extending full height (approximately 360 ft) to the dam crest (elevation 1257 ft) which permitted debris removal equipment access to the trash structure under all reservoir conditions was initially considered. With such a design, intake tower entrance area was not of significant concern. However, structural and foundation concerns resulted in consideration of a shorter tower which would not provide accessibility during all reservoir conditions. Reservoir routing studies which incorporated the estimated intake tower blockage conditions previously discussed were accomplished to develop a tower design concept which would (a) provide sufficient capability to essentially meet the project authorizing parameters for the project design flood condition and (b) minimize reservoir drawdown time following spillway design flood conditions while considering that debris removal equipment access could not be provided at all reservoir elevations. A further constraint was that the differential pressure across the intake tower trashrack members could not exceed 50 ft for structural reasons. These studies indicated that a 50-foot diameter tower with top elevation 1100 feet (approximately the 20-yr recurrence interval regulated reservoir elevation), the outside perimeter of which would essentially be a large trash structure, was required. The primary debris removal operations for the normal type flood events will be accomplished from a deck located at the 960 ft elevation. However, a vehicle bridge has also been incorporated into the design to provide debris removal equipment access to the top of the tower.

Tunnel Entrances. The hydraulic design of the 9-foot tunnel entrance and modifications was relatively uncomplicated. The entrance size and control gate geometry is identical to that which has performed successfully over the past 40 years. The 23-foot tunnel modifications; however, are hydraulically complex. Dual entrances are provided to permit flexibility in operation and maintenance of the tunnel and to provide discharge capability in the remote event that one control gate is disabled. Although the design discharge of the 23-foot tunnel is 13,000 cfs, each entrance was sized to ensure that at least 75 percent of the required total flood control discharge of 17,600 cfs could be provided if any of the entrances to either the 9- or 23-foot tunnel were inoperable. Two 7-foot-wide by 11.5-foot-high entrances provide this capability. The entrance invert elevations of 910 and 925 feet, which are lower than the existing entrance elevation of 970 feet, were selected to assist in minimizing operational constraints which presently create adverse environmental conditions relating to passage of sediments downstream during periods of fish spawning. The invert elevations; however, could not be set so low as to permit large volumes of heavy sediments to be passed through the 23-foot tunnel at the high velocities which will exist. The entrance geometry was designed based upon simple elliptical curves similar to the short-skewed design that has proved satisfactory at the Corps of Engineer's Dworshak Dam in the state of Idaho. Sizing of the air vents to the gate area was based upon prototype air demand test results from the Seattle District's Libby Dam.

Tunnel. The horizontal curve initially designed to connect the new and existing portions of the tunnel was a simple, 500-foot radius curve. The physical model operation indicated that acceptable hydraulic performance would exist even with a shorter 250-ft radius curve. The shorter radius curve results in a significant savings in construction costs and time.

Exit Structure. The tunnel exit structure is designed as a curved, directional flip bucket to direct the high velocity exit jet away from impingement on the rock face of the channel wall immediately across from the exit portal and into the confines of the downstream tunnel. This design is necessary to minimize mortality to downstream migrating fish which will be passed through the 23-foot tunnel. The entire exit transition is located in the downstream 50 feet of the existing valve concrete plug area which requires removal in conjunction with the valve removal work. Therefore, additional tunnelling to accommodate the exit curvature will not be required.

Abstract
Flooding Resulting from Typhoon Uring in Ormoc City,
Leyte Province, The Philippines

by Monte L. Pearson, USAE Waterways Experiment Station
and John G. Oliver, North Pacific Division, Corps of Engineers

Typhoon Uring passed over the Island of Leyte on 5 November 1991 on an east to west track north of Tacloban City and Ormoc City. The center of the typhoon was at the midpoint of the Island about 10:00 a.m. and took 3 to 4 hours to pass across the Island width. Widespread damage from flooding was left in its wake. The Coastal Plain from Ormoc City at the north to Baybay at the south situated at the western side of the Island received major damages. The Ormoc watershed and Ormoc City were the hardest hit areas.

The storm speed and the accompanying intense rainfall were the main causes of the damages. Because of the storm's speed on the eastern slope of the mountain range, low elevation runoff passed through the Coastal Plain before higher elevation runoff was routed to the coastline. Conversely, on the westside the higher elevation runoff arrived at the coast about the same time that lower elevation runoff was most intense.

Rainfall intensities were extremely high with 140.2 mm recorded at Tacloban City in 3 hours, 580.5 mm and 350.0 mm recorded in 36 hours at PNOC Raingage 1 and 2. The major portion of the rain at PNOC is reported to have occurred in 3 hours. Extremely intense rainfall was also reported to have occurred at Ormoc City in a 3-hour period.

Soils in the region were totally saturated a short time after inception of intense precipitation. Soil strengths were decreased and significant surface failure occurred to a depth of about 1 to 3 m in the upper basin. Stream bank and bed erosion was also intense. Bulking of flows by sediments contributed to the magnitude of the flood. It is also likely that the upper river basins both east and west experienced debris flows that at lower elevation dropped to intense sediment transport leaving the larger boulders but continuing to carry woody debris and up to gravel size material that was capable of damming bridge sections.

Streams in the region are well incised, and side slopes have limited stability. Evidence of high sediment transport prevail throughout the systems. Stream lower reaches are braided at low flow and have numerous channels. Stream bed slopes are high, estimated to exceed 1 percent within 200 m of their mouths and average 5 to 7 percent throughout their lengths on the western slopes. Under those circumstances, stream orientation under various flows is difficult to predict. Deep erosion around hardened surfaces, bank protection, bridge piers, abutments, and contractions are normal. Large fluctuations in river bed elevations are common as sediment pulses pass through reaches.

The intense flood at Ormoc City which lies just below the confluence of the Antilao River and Malbasag River was caused by extremely intense rainfall and upper basin soil instability. The major loss of life at Ormoc City

occurred just upstream of the main City, in the Isla Verde area. Flood plain zoning may have assisted in reducing the losses experienced.

Factors that could reduce future localized flooding and infrastructure damage are improved approach alignments of the streams with bridges, reduction of stream contraction, and raising of bridge decks to prevent damming of bridge section with debris. Based on the 5 November 1991 flood event and a future risk analysis, bridge redesign criteria development throughout the system should give consideration to river alignment, existing river widths and the latent instabilities associated with high velocity major sediment and debris transporting systems.

The following is a list of recommendations.

1. Provide zoning to prevent habitation in high risk areas.

2. Provide flood warning systems where practicable.

3. Establish the frequency and size of the event so both risk and the economic impact of remedial measures can be analyzed.

4. Improve design criteria for infrastructure.

5. Stabilize slide surfaces to minimize sediment mobilization during minor events.

6. Provide stream improvement and diversions.

Background

The Philippines is an archipelago of 7,107 islands and stretches from the south of China to the northern tip of Borneo. Total population is about 60 million. Leyte is one of the major islands and lies at about 11° 15′ N latitude (Map 1).

Leyte Province is located on the Island of Leyte in the Republic of the Philippines. The island is 350+ km south southeast of the capitol city of Manila. Typhoon Uring moved onto the island early Tuesday morning on 5 November. Although storm winds were relatively weak, packing sustained winds of only 55 kph, it unleashed heavy rains over Leyte beginning on 4 November on a Monday night. The rains became extremely intense Tuesday morning. Landslides in the mountains were triggered and vast parts of lower elevation areas of western slopes were flooded by mud and water by noon Tuesday.

The flooding is considered to be the worst to have occurred in the Philippines in 7 years. Up to 3 m of soil laden floodwater submerged Ormoc City and outlying towns. Coastal area residents were caught without warning. The death toll was extensive with most being drowned or buried in mud as their houses were swept away. The city of Ormoc with a population of 150,000 was hardest hit with approximately 4,800 dead and 1,857 missing.

Bridges along the western coastline received severe damage. In total, 16 bridges were damaged or destroyed. Most of this occurred between the cities of Ormoc and Baybay. Early estimates of property damage have been P 395M. Agricultural crops, poultry, and livestock accounted for P 35M, infrastructure, P 130M and private properties, and public facilities accounting for the remaining P 230M. Power failure was widespread as power poles were destroyed.

Mission

On 12 November 1991, Headquarters, US Army Corps of Engineers was contacted by the Department of State regarding Corps assistance on determining the cause of the flooding and possible mitigation measures. Two engineers with extensive experience in river morphology were dispatched to the Philippines on 4 December 1991. The team was comprised of Mr. John G. Oliver, Chief, the Hydraulics and Civil Design Branch, North Pacific Division, and Dr. Monte L. Pearson, Senior Research Scientist, Geotechnical Laboratory, US Army Engineer Waterways Experiment Station.

The team spent 5 December through 13 December 1991 in the country. Meetings were held with the US Agency for International Development, and primarily with the Philippine Government Department of Public Works and Highways. The basic requesting letter, complete listing of contacts, and basic trip itinerary are included in Appendix A.

Geologic Setting

The Island of Leyte was formed by volcanic action consisting of strata volcanoes, dome complexes, pyroclastic/tephra cones calderas and compound volcanoes (Philvocs Annual Report 1988). The volcanic core complex is concentrated on the north central portion of the island. The central areas are classified as "volcanic terrain."

The Philippine Fault System is the major tectonic feature of the region. The system trends NW-SE through Leyte with all of the volcanic cones resting on the eastern block of the Philippine Fault. The intense tectonic activity has highly sheared and fractured all geologic formations on Leyte.

The basement geology of Leyte is pretertiary igneous and metamorphic rocks traversing the length (North-South Axis) of Leyte. Fluvial marine and terrace gravel deposits of early Neogne-Latte Preleogene overlay the basement complex. Pleo-Pleistocene volcanic formation of andesitic composition is the youngest exposed formation within the area. The flanks of the volcanic complex are blanketed by pyroclastic materials mainly of Lahar origin. The Lahar deposits in the watershed area are typical poorly sorted (boulder-pebble-gravel sized andesitic clusts) and matrix (sand size). The lowlands are fluvial sediments of unconsolidated matrix gravel type.

The soils in the Ormoc watershed have been classified by the Philippine Bureau of Soils as "upland soils." They are characterized by undefined soil horizon with great erosion potential. Soils are formed originally from decomposed andesitic rocks. These are granular and noncohesive, unstable and highly susceptible to erosion and transport. The upper watershed has steep slopes, and a high rate of soil formation due to rapid decay of andesitic rock materials and climatic factors. In order to maintain slope equilibrium an equal volume of soil mass removal and reformation occur (regolith). The numerous fresh slide in Bao, Malitbog, and the other drainage included in the Ormoc watershed support this data. High rainfall rates serve as a catalyst which triggers the majority of the shallow mass movements. The Department of Science and Technology (Department of Science and Technology 1991) also supports this finding, and it states that the average soil profile is 7 m thick.

Drainage System

The Regional Disaster Coordinating Council of Region 8 (Regional Disaster Coordinating Council 1991) provides a complete description of the Ormoc watershed. Ormoc watershed is composed of three major subdrainage basins. This report will only describe the northwestern two drainage. They are the two major systems directly associated with the 5 November 1991 flooding of Ormoc City.

The Antilao and Malbasag Rivers are the two major drainage systems that directly impacted Ormoc City. These two rivers converge upstream of Ormoc City and the Isla Verde Area (Map 2).

The Antilao River drains the northernmost portion of the watershed and is composed of three subbasins (Map 3). The middle portion of the watershed is drained by the Malbasag River and only has one subbasin. The Malbasag River is the smallest of the two in area and channel length, 10.8 km compared to 16.3 km for the Antilao drainage (Figures 1 and 2). The total drainage length of the two systems is approximately 27.0 km.

Drainage in the Ormoc watershed is dendritic in pattern and well incised. Upper channel incisions are characterized by a 1/3-width/depth ratio based on a ridge/stream measuring system. Progressing downstream the width/depth ratio just upstream of Ormoc City is about 3 to 1.

The Antilao River has a vertical drop of 84.5 m in 13.2 km on the mainstem for an average of 6.4 percent slope, whereas, the Malbasag River has a slope of 6.2 percent. The average fall of both streams is 64.8 m/km. All the streams in the Ormoc watershed flow southwest and converge above or near

Ormoc City (Map 2). The confluence of the two rivers is 2.5 km from the Camotes Sea. The junction is about 5 m above average sea level (ASL). The mainstem is 13 to 15 km in length and drains approximately 190 km^2.

Data gathered during field reconnaissance indicate that mass movements were shallow failures ranging from 1 to 3 m in depth (Photos 1 and 2) and 50 to 100 wide at head failure zone. Movement generally occurred from ridge line to the channel bottom. The soil mass in the upper watershed has been classified as cohesionless media which failed at a ratio of length/depth to shear plane of 10 to 100. Photos 1 and 2 provide positive illustration of this relation.

In a dry state these cohesionless soils rely upon interparticle frictional strength for stability. Upon wetting, the interparticle frictional strength is reduced, and as total saturation occurs, the strength factor is reduced to 0 at which time failure occurs. Further, the highly weathered and relatively thick soil masses have developed internal shear planes. Subjected to intense rainfall, the shear planes become failure planes. The short and high intensity rainfall in these circumstances created mass movement features that were long and shallow. Combining all the data, surface soil mass with internal shear planes, stream side slopes exceeding 60 percent, and cohesionless soil, it is apparent that during wet conditions the slopes in the upper 2/3 of the Ormoc watershed are highly unstable.

Channel Morphology

For descriptive purposes the Ormoc drainage basin has been divided into three basic valleyway-channel geometries (upper, middle, and lower) that are directly related to the geomorphology of the region (both slope and channel processes).

The upper basin area represents the areas of highest topographic relief drainage and are the headwaters for the basins. Down-drainage out of channel topography has slopes up to 30 percent. Side slope in the valleyway channels are up to 60 percent. According to Land Resource Evaluation Report (LRER) Leyte Province (Bureau of Soils and Water Management 1986), this area constitutes 20 percent of the watershed area. Channelways are deeply incised with near vertical channel walls up to 10 m above the channel bed (Figures 2-3 and 9-10). Large historical mass movement scars are visible with recent (5 Nov 91) small failure scar superimposed. The dates of historic failures are unknown, but dating could be used to aid in establishing flow frequencies.

The width/depth ratio normally related to change geometry has been modified to represent channelway and channel area. The channelway is the deeply incised area with the active channel (braided) in the basin area. The upper basin area width/depth ratio is nearly 1 to 3. This portion of the watershed has experienced the highest percentage of mass movement events. The majority and most recent are shallow failure features which normally ran out to the channel area at the base of the slope. Material delivered by this process is saturated. Upon delivery to the stream, materials are immediately entrained and the total flow is bulked by the added sediment. Flow bulking by sediments was a significant phenomena during the 5 November 1991 storm in the higher and steep channel slope areas. The sediment-laden water was transported down the deep, narrow and generally straight channel system to the Middle Basin Area (Photo 3).

The Middle Basin Area terrain has slopes of 18 to 30 percent and represent about 18 percent of the total basin (Bureau of Soils and Water Management 1986). The width/depth ratio changes to approximately 2 to 3 and the channel side slopes are still steep at 60 percent (Figures 4-5 and 11-12). Again, shallow mass movement features are prevalent along the channel valleyway. Sediment loading and bulking processes were similar to that in the upper basin area. With the increase in discharge, bed degradation and bank erosion processes produced additional sediment to the system. The channel in plan view at the low flow is more meandering and braided. At higher flow this meandering and braided form undoubtedly disappears.

The lower basin area (excluding Ormoc City and the Delta Area) is still well incised with a width/depth ratio of about 3 to 2. Meandering and channel braiding are the major low flow plan form features. Down-drainage slopes are reduced to 8 to 18 percent.

The mainstream of the Antilao and Malbasag Rivers, as stated earlier, are incised and highly meandering in this reach (Figures 6-8 and 12-13). The meanders induce a high degree of channel sinuosity, which create deposition zones upstream of each meander bend (Photo 3). Momentum loss and backwater effect results in deposition of the boulder/gravel material in transport.

Loss in momentum, produced by the sharp direction change, reduces stream power which reduces transport capacity. Once the flow has exited, the meander stream power is regained, and bend, bed, and bank erosion/scour occurs. This sedimentation/erosion process repeats itself through the lower reach. With each repetition the D_{50} size of material transported is reduced. This process explains the lack of a large volume of boulder to gravel size sediments within the flood effected area of Ormoc City. The gravels and finer fractions did continue down the system and deposited in the city and Camotes Sea. Significant amount of woody debris was also transported and may have formed up as part of a debris flow at the front of the flood (Photo 4).

Ormoc City and Delta Area

The Ormoc City/Delta Reach can be considered the delta/beach reach of the system. Town development and cultural features have fixed the channel location. The junction of Antilao and Malbasag Rivers just upstream of Ormoc City creates a single major river through town. The channel width/depth ratio through town appears to be 3 to 1; however, the incision and width are not natural. The channel geometry has been adjusted and the reach is smaller than those of the lower basin areas. The reduction in flow area and adjustment of slope associated with the delta are contributing factors which resulted in massive out-of-channel flow in the Ormoc City area. Photos 5 and 6 show that at the Antilao Bridge (km 1010+968.2) a 90° direction change occurred. The momentum change combined with the contraction should have had the severe effect of putting initial flow overbank at this location. Along with debris flow and woody material impacting this area, the bridge was eventually removed. Flood water entered the Ormoc City street system to a depth of 3 to 5 feet, creating significant property damage. The recent lobate deposit at the mouth of the system suggests that the coarse fraction of sediment continued to transport in the channel and through the town to Ormoc Bay. The Isla Verde area upstream of the Antilao Bridge was flooded several feet above overbank, and residents living within the floodplain were decimated (Photo 6).

Ormoc City is located at the lowest elevation on the coastal delta of the Antilao and Malbasag Rivers. Both drainage join about 2.5 km from the coastline and 1 km north of Ormoc City Proper. At the confluence point the drainage is wide. Significant residential development within riverbank lines and on banklines between Ormoc and the confluence point had occurred prior to the flood.

Highway No. 302 channelizes the drainage system to the northside of the highway. The emplacement of Antilao River Bridge induces a 90° channel bend. At the bridge site the channel changes from 300± m wide by 3 to 4 m deep to 30 to 40 m wide and 10 m deep. The change continues from the river bridge through Ormoc City to the bay. The constriction greatly increases the flood potential immediately upstream and through the City (Photos 4, 5, and 6, Map 2).

Upstream of the Antilao River Bridge in the Isla Verde area (Map 2 and Photo 5) is a zone composed of a colluvial sediment of gravel and sand size. The material is representative of the deposition area of steep braided gravel channels. As stated earlier, this section of the floodplain was densely inhabited. Based on information provided by the Ormoc City Engineer, there were a variety of dwelling types; none able to withstand a significant flow event.

A third drainage system (Biten River) flows just to the east and south of Ormoc City Proper. The extent of flooding or impact on the Antilao and Malbasag Rivers flooding of Ormoc City was not determined but could have been a factor.

Meteorological Conditions

Tropical Storm Uring developed on 2 November 1991 and continued until 6 November 1991. Uring was a relatively small and weak storm system. Uring maintained typhoon status for less than 24 hours starting about 1000 hours on 5 November 1991, while located some 350 km east of the region. The storm tracked east to west across Leyte passing north of the Ormoc City Area. After crossing Leyte, Uring weakened to a tropical depression and on 6 November 1991 at 10:00 am, tropical depression Uring dissipated to a low pressure system (Map 4).

As the storm system advanced on Leyte, intense rainfall started about 0730 hours on 5 November 1991 at Tacloban City (Map 4). Rainfall records at the Tacloban Airport indicated 140.2 mm of rainfall in a 24-hour period with the most intense occurring in only 3 hours, 0730 to 1030 hours.

Intense rainfall began about 0830 hours at the PNOC rain gages 1+2 (Map 3). At about 1130 hours the highest rainfall intensity had decreased or stopped according to PNOC officials. Uring was traveling at 12 kph as it moved across the Island. Based on travel speed and distance, intense rainfall should have started in Ormoc City at about 0930 hours on 5 November 1991. As per conversations with city officials, intense rainfall did occur approximately 0930 hours on 5 November 1991. Rainfall intensity and winds were of a high magnitude; areas of extensive blowdown occurred. Photo 7 shows an area of blowdown in the upper drainage section of the watershed.

Rainfall records (Table 1) from November 1976 to 1991 for the VISCA weather station located 8 km north of Baybay, Leyte provides a general indication of rainfall quantities for the coast areas period of record. Baybay is located 30 km south from Ormoc City and some 850 m lower in elevation than the upper portions of the Ormoc watershed. Based on meteorological principles, it is possible to extrapolate that rainfall amounts in the upper watershed areas could commonly be one or two orders higher in magnitude. The VISCA station reported 238.4 mm in 24 hours compared to PNOC Rain gage No. 2 of 580.3 mm (the PNOC Hostel and nearest to Ormoc watershed, Map 3). This rain gage is located at an elevation of 435 m above sea level which is only 2/3 up the drainage basin in relative elevation.

The Flood

At Ormoc City the flood waters flowed out of the lower basin into the Isle Verde area entraining buildings and most other items in its path. It is probable that a front wall of woody material and debris reached the bridge and restricted flow through the Ormoc City channel. The bridge restricted channel width and 90° bend created some backwater effect almost instantaneously. Channel width at the available slope was inadequate to contain the flow within banks. The stage hydrograph of the event as described by local residents and partially recorded on video tape is as shown in Figure 14. The water surface rose by 7 feet in 15 minutes, and an hour after flow peaked, the hydrograph was falling. The total flood lasted less than 4 hours. Flow in the streets of Ormoc City as recorded on video tape had a depth of 3 to 5 feet. Average sediment depth was described as 2 feet deep after the flood waters had receded. Sediments in the streets were fine grained and characteristic of suspended load. Materials offshore of the river mouth appeared to have a much greater fraction of coarse material, and it is believed that most of coarse material passed through the river channel as near bed suspended load. Sediment transport and flow should then be governed by principles used for Newtonian fluid.

The Special Task Group Regional Disaster Coordinating Council, Region 8, used the rainfall record of 580 mm at Tongonan, PNOC Rain gage No. 2 which is nearer and more proximate to the Ormoc Watershed and assumed that 80 mm of the precipitation occurred between 10 p.m. on 4 November to 8 a.m. on 5 November 1991. Based on that assumption, 500 mm was the rainfall from 8 a.m. to 11 a.m. on 5 November 1991. The total volume of water that flooded Ormoc City was:

watershed area x rainfall
4,500 hectares x 500 mm or
45,000 sq m x 0.5m
Q = 22,500,000 cu m of water

We estimate that about an equal amount of sediment was transported. Assuming a 35 percent porosity in sediments, the total volume of fluid was about 37,000,000 cu m. To estimate the peak flow rate and to get further insight into flood potential at Ormoc City, a preliminary routing was done on the flood. The basin was broken into 6 areas with 500 mm of rainfall introduced into areas 3, 4, and 5 uniformly between 0800 and 1100 hours on 5 November and in areas 1, 2, and 6 between 0900 and 1200 hours (Map 5). Sediments were introduced in run No. 1 Figure 15a at 0800 hours and in run No. 2 Figure 15b at 0820 hours. Sediment was only introduced in areas 3, 4, and 5. Sediment introduction was in proportion to rainfall. The flood peak at Ormoc City was at 1050 hours for run No. 1 and at 1120 hours with a 10-minute shift in initial sediment input on run No. 2. Peak flows were between 70,000 cubic feet per second and 80,000 cubic feet per second, channel capacity through the City is estimated to be much less than 30,000 cubic feet per second. The receding side of the hydrograph on both runs is extremely sharp which is the result of the assumptions on where sediment was introduced and upon rainfall intensities.

An inspection of the routings indicates that the slope of the rising leg of the routed hydrograph is too flat, and it is likely that a debris flow was at the front of the actual flood. The debris flow would have retarded the arrival time of the peak because of increased viscosity and friction. It would also have increased the peak flow. The receding side of the hydrography is too steep, based on the local description of the stage hydrograph. Lower elevation sediment entrainment was significant and was not introduced into the simulation, and some ponding occurred in the Isla Verde area. Therefore, a gentler slope would be expected if more realistic assumptions were made.

From the limited analysis, it is evident that the sediment event accompanying the rainfall had a significant influence. Detailed analysis of the hydraulic conditions at Ormoc City could be used to confirm and adjust the routing discharges. The more critical variables could then be reentered into a flood routing program. Variations in the sediment entrainment and routing scenarios could then be used to establish variance in risks associated with different rainfall events.

Bridge Damage

Bridge damage along Highway 302 between Ormoc City and Baybay was extensive (Map 6). Bridge damages at Ormoc City and in other drainage are the result of over constriction of the stream at bridge crossings and poor alignment. Loss of bridge approach control structures, erosion around piers, deck uplift by debris and loss of abutment fill by piping were the common modes of failure (Photo 8). Most of the damaged bridges observed had constricted the river width by 50 percent or more. All bridges appeared to be constructed with near river bed spread footings at piers and gravity section abutments. Approach controls were grouted riprap with little if any toe burial and did not appear to always reach top of bank.

There were numerous cases where the banklines had eroded behind the bridge abutments. It is speculated that bridge constriction created a high differential head across the approach fill. Approach fill fines were piped out and a channel developed. Increasing the bridge length would help alleviate the problem under similar flood circumstances.

Other bridges failed because of river bed erosion below pier or abutment toes. Deeper footings, pile supported footings, and longer bridge sections in those cases would be beneficial.

River flow alignments appeared to be a major problem at bridge crossings. The alignment problems were, prior to the flood, partially corrected by grouted revetments.

Conclusions

The combination of topographic, hydrologic and physiographic features on the Island of Leyte leads to rainfall, sediment loading, runoff, and flood problems somewhat unique to steep, short, unstable drainage basins. Western slope drainage basins are more prone to intense runoff than are east slope drainages because of the higher average stream gradient. Flood frequency and net runoff during even modest severe rainfall events are dependent upon the cumulative effects of basin geology, topography, hydrology, and the geomorphic and antecedent moisture characteristics of the drainage basin prior to the event. Rainfall events of similar magnitude may result in very different flooding characteristics depending on event sequencing, antecedent moisture conditions in the basin and residual soil strengths. The flood of 5 November 1991 appears to be a product of event sequencing, rainfall intensity, and soil instability. In the 24 hours prior to the event, the basin had been subjected to significant precipitation. Landslides triggered mud and debris flows during the storm event. Moisture conditions in the drainage were high due to previous rains, and shortly after intense rains associated with typhoon Uring began, soils were totally saturated. Therefore, flow concentration times were short and side slopes were weakened to the point of failure. High precipitation bulking and perhaps the effect of the more viscose mud and water mix on channel roughness culminated in flow depths greatly exceeding channel capacity on the alluvial fan. The peak flow from the event was on the order of two times the flow that would have been expected if major sediment entrainment had not occurred.

Recommendations

The loss of life at Ormoc is considered the most severe impact of the flooding on Leyte. A significant reduction of this impact can be obtained by preventing habitation in high risk areas. The delta of the Antilao and Malbasag Rivers immediately upstream of Ormoc City Isla Verde Area was densely inhabited prior to the flood. Most of the deaths were among the residents of that particular area. It is now being resettled. Immediate action on zoning to prevent that rehabitation and enforcement of the zoning can go a long way toward mitigating a future disaster.

Another method of reducing loss of life in somewhat lower risk areas than the delta region are flood warning systems. The western slope of Leyte where Ormoc is located is probably not adaptable to this because of the short, steep drainage. Eastern slopes are however well situated for flood warning systems.

Flood control storage systems are measures used elsewhere to control runoff from major events, and have been mentioned in several of the Philippine Government agency reports. Onstream storage does not seem practicable at first assessment. The high sediment yield, steep stream gradients and incised nature of the channels normally make economic development of onstream storage difficult. On the western slopes, offstream storage also appears to be limited by topography. The eastern side may however benefit by offstream storage. Rice paddies and other natural impoundments may already be effective in attenuating major floods.

The frequency of an event is an important factor in determining risk and economic impact. Based on the memory of the population, the most recent flood prior to 5 November 1991 occurred in the 1930's. Rainfall records observed also indicate that the intensity of the 5 November 1991 event was the greatest in the 19-year period of record. The frequency and the magnitude of this and other events should be determined if economics is to be the basis for costly changes in design criteria for infrastructure or major flood control works. Zoning should also incorporate some logic on risk and risk assessment which depends on flood frequency and magnitude.

Numerous streams flooded during the Uring Typhoon event. Measurement of high flow marks, estimates of sediment yield by quantifying slides mass, and hydraulic analysis to establish peak flows combined with flood routings can establish the magnitude of the event. Event frequency may be more elusive as it is believed that it is a function of length of precipitation, intensity of precipitation and the slope stability of the basin at the time of the event. Methods that are used include population interviews, historic landslide analysis, storm frequencies including hindcasts and historic flow measurements. A combination of methods will probably be required in this situation.

Design criteria for infrastructure (i.e. bridges, revetments, power poles, and other items) appear to be based upon fairly modest climatic conditions and upon more tranquil drainage systems. A review of bridge design, river mechanics and international experience may indicate that a change in the design criteria could improve the life-cycle costs of the infrastructures.

The presently active slide areas will continue to yield sediments to the streams at a fairly high rate until naturally revegetated or otherwise stabilized. A storm of lower intensity and water content could produce a higher flow than Typhoon Uring under these circumstances as sediment bulking of the flow could be more pronounced. Steps taken to revegetate and stabilize the slides would reduce the period of risk.

Stream alignment improvements at bridges, hydraulic improvements through populated areas, or diversions around populated areas are possibilities for minimizing impacts. Extensive engineering analysis is required to define the benefits of such options.

Bibliography

Bureau of Soils and Water Management. 1986. Land Resources Evaluation Projects, Republic of the Philippines.

Department of Science and Technology. 1991. Scientific Assessment Report: Ormoc City Flood on 5 November 1991, Ormoc Task Force Scientific Study Group, Department of Science and Technology, Republic of the Philippines, 27 November 1991, p 22.

Ormoc City Engineer. 1991. A Pictorial on Ormoc City Disaster, 5 November 1991, DENR, Region 8, Republic of the Philippines, p 17.

Regional Disaster Coordinating Council, Region 8. 1991. Investigation Report of the Ormoc City Disaster, B/Gen Vicente S. Garcia, Jr., Chairman, Republic of the Philippines, p 66.

Second Leyte Engineering District. 1991. Pictures of Damages caused by Typhoon "Uring" and Flash Flood 2nd LED Ormoc City, 5 November 1991, Republic of the Philippines.

Yolo, Vicente A. Jr., (Submitted by). 1991. Typhoon Damages Reports with Pictures Caused by Typhoon "Uring" Occurred on 5 November, 1991, and Temporary Restoration Work, 3rd Leyte Engineering District Baybay, Leyte Republic of the Philippines.

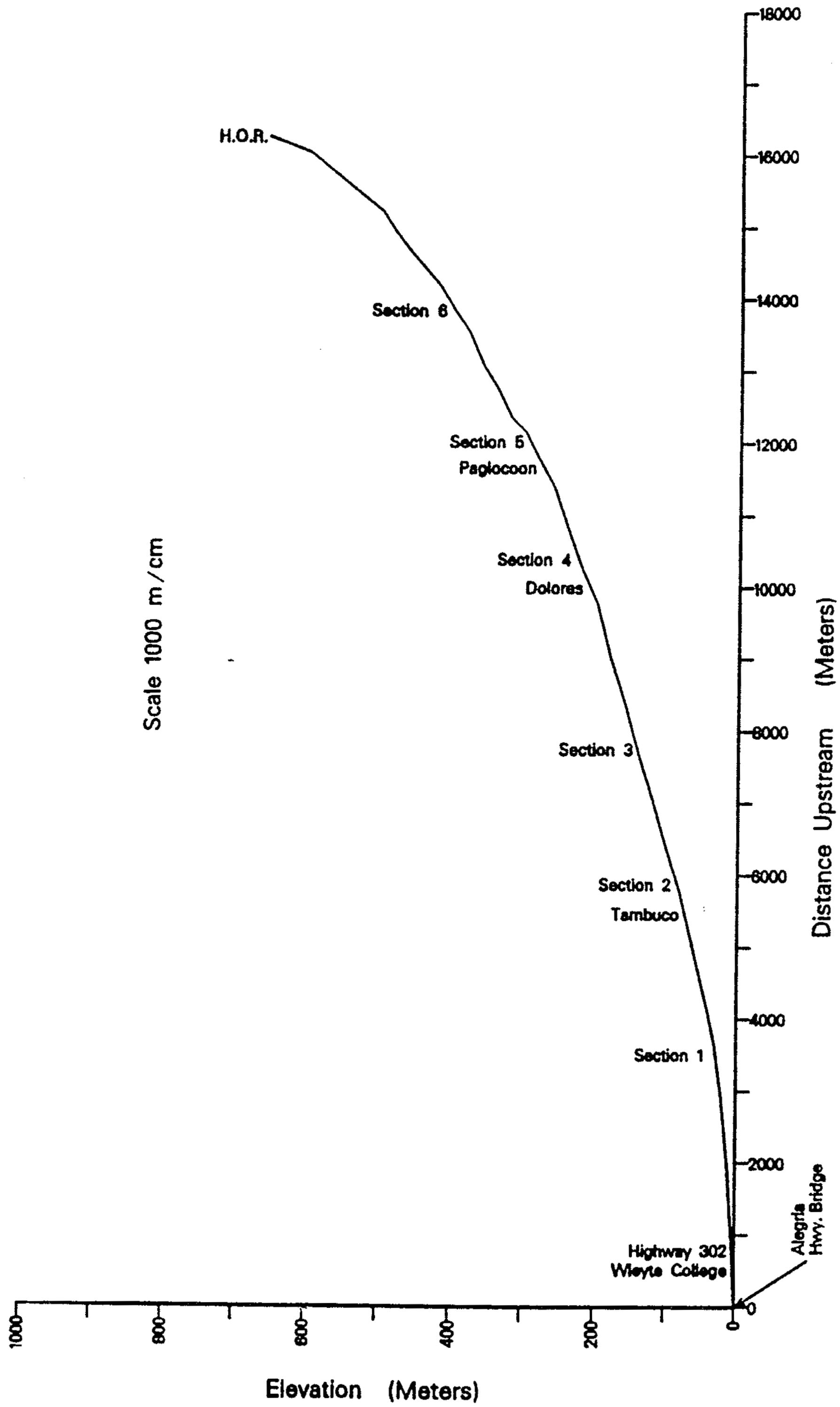
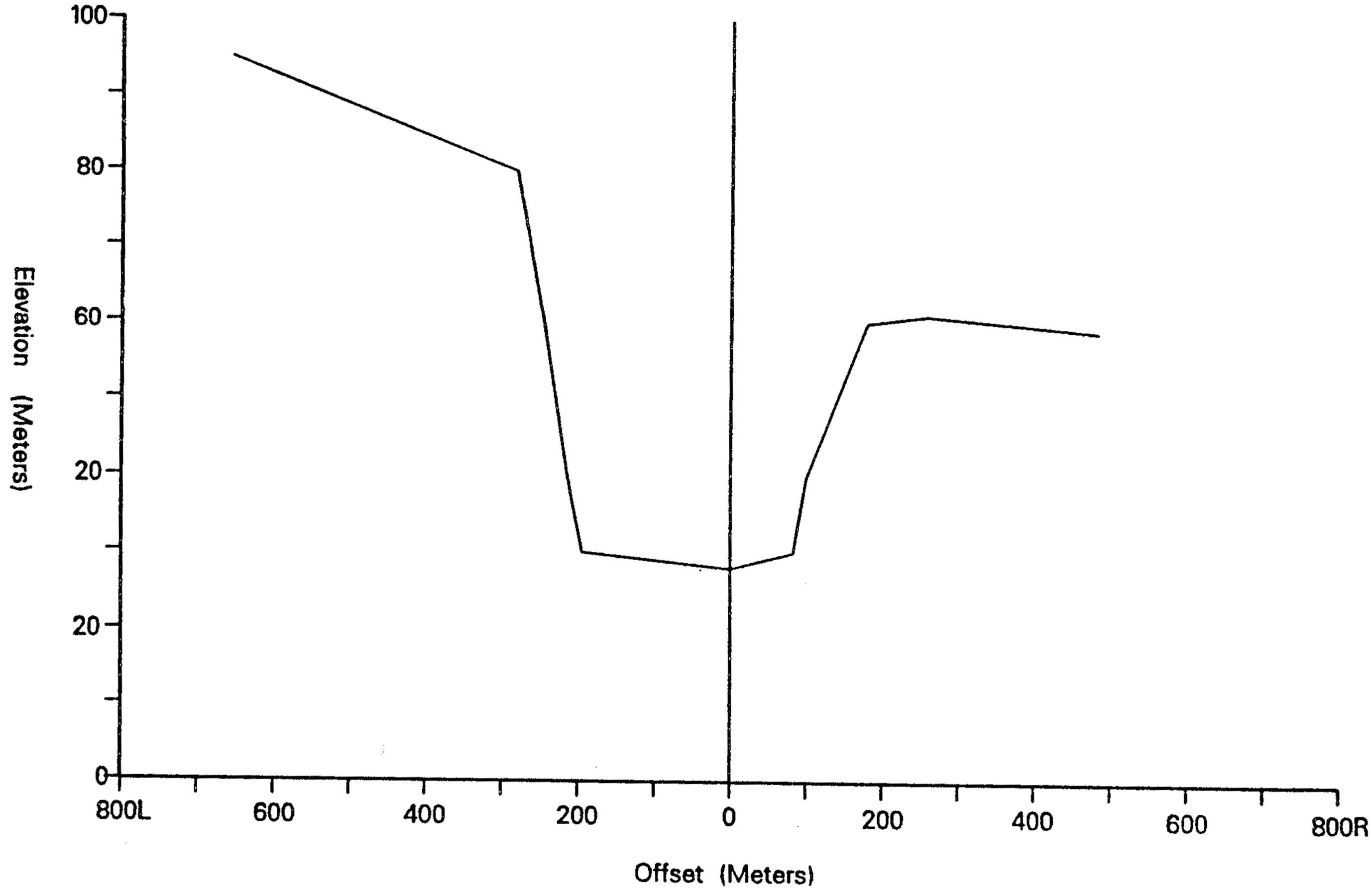
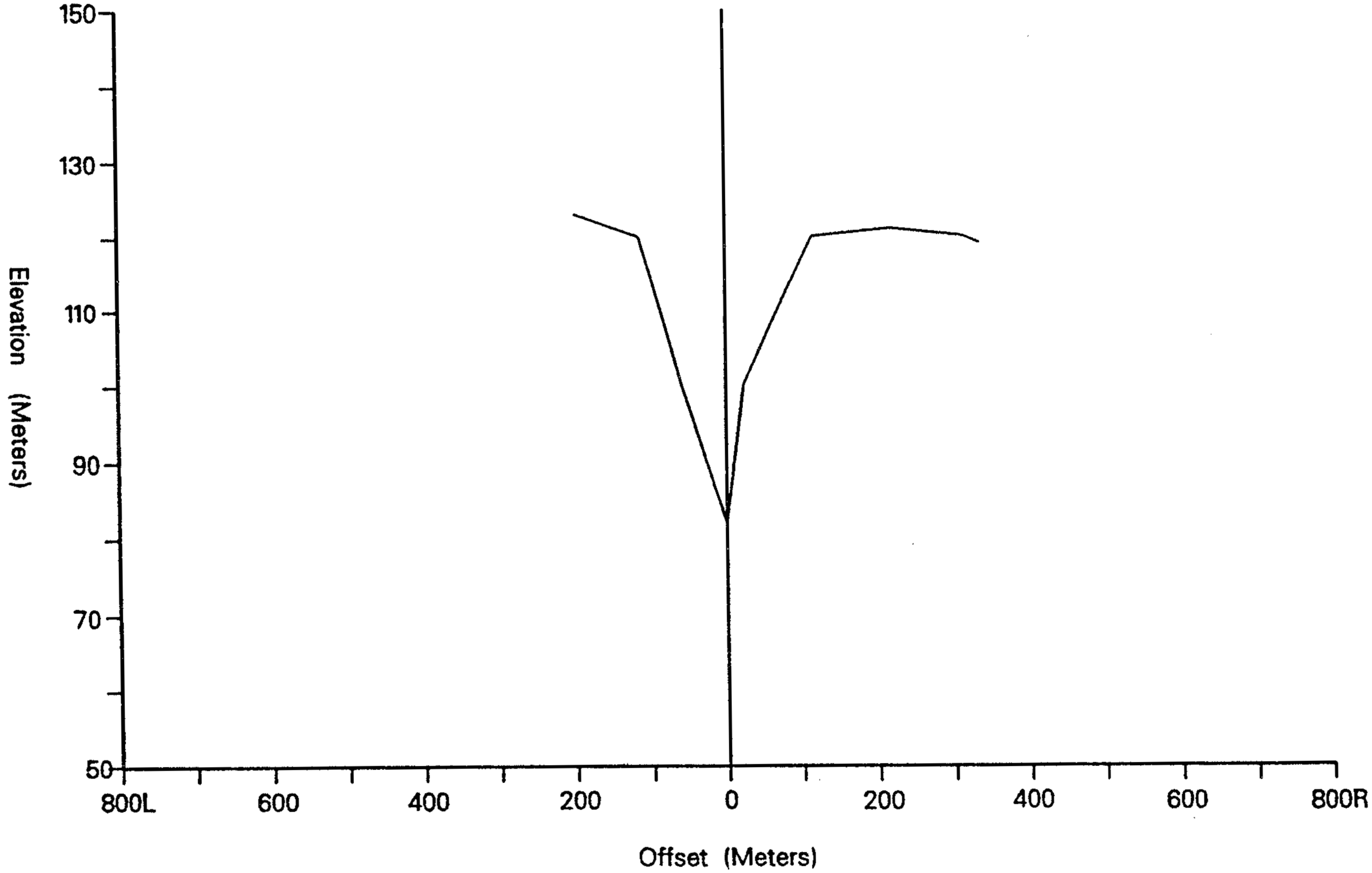


Figure 1. Antilao River Channel Profile and Cross-Section Stationing



Section 1
Sta. 35 + 14.00
Scale 100 m /cm

Figure 2. Cross Section 1 Antilao River



Section 2
Sta. 58 + 83.18
Scale 100 m /cm

Figure 3. Cross Section 2 Antilao River

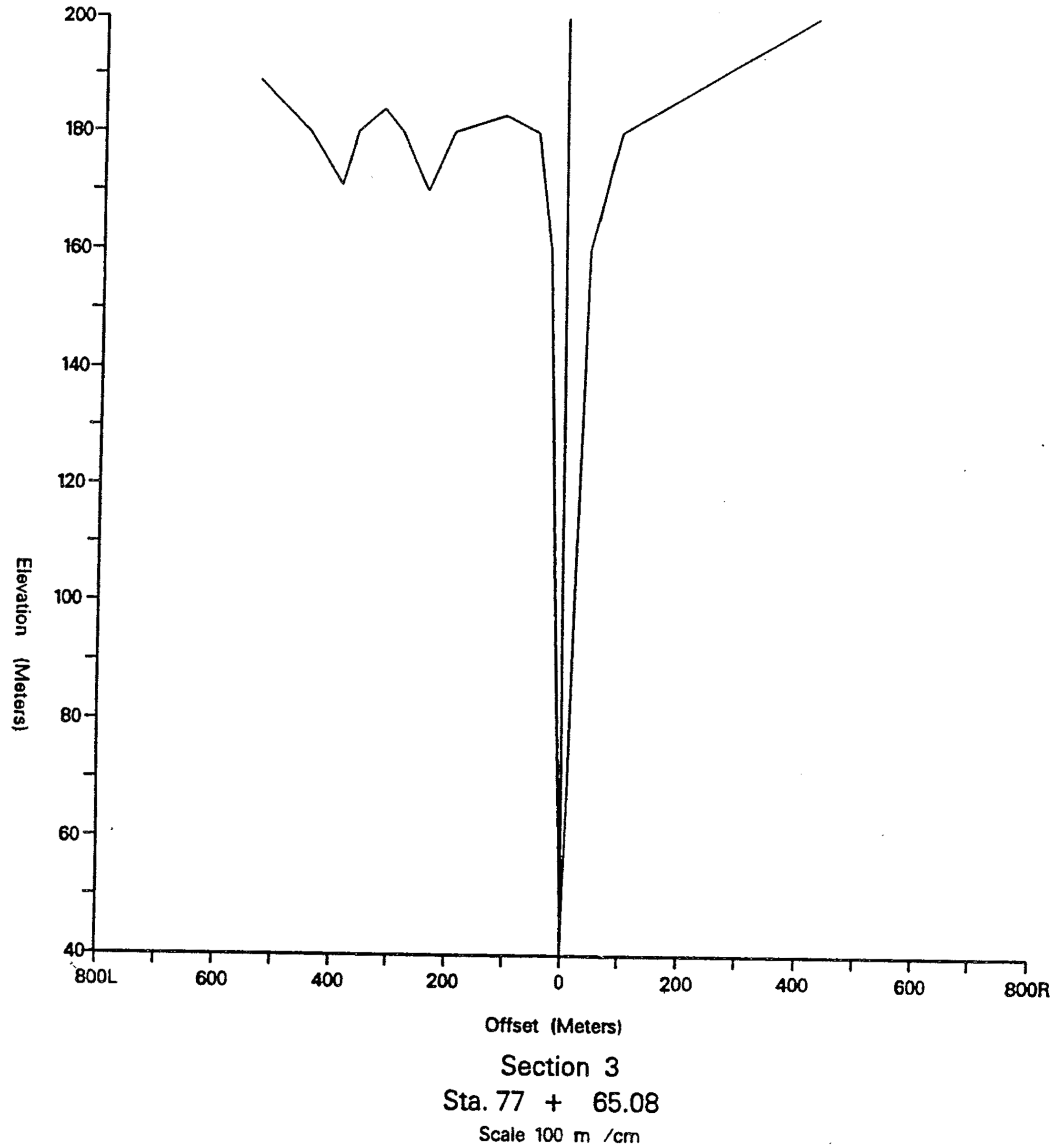
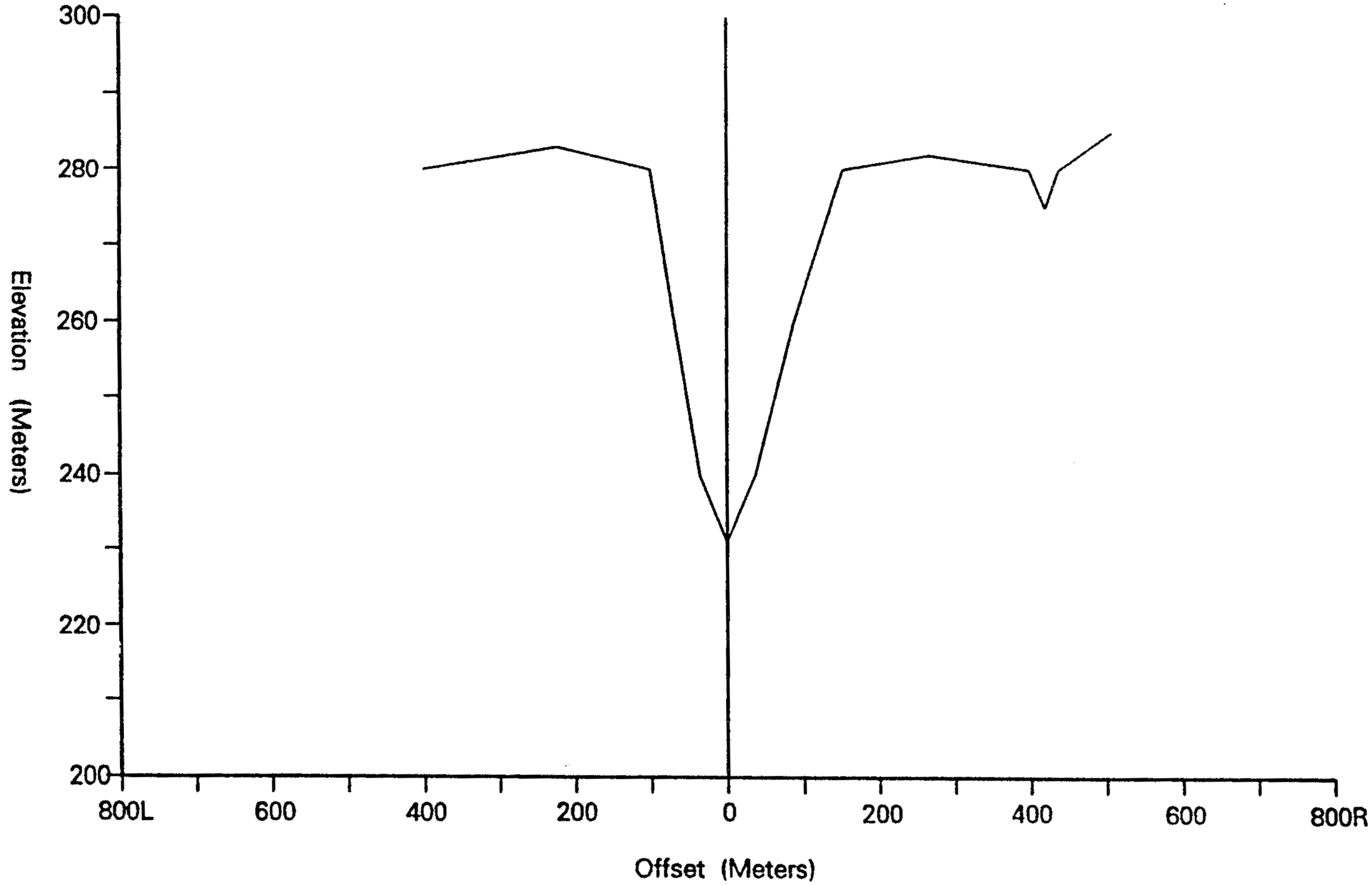


Figure 4. Cross Section 3 Antilao River



Section 4
Sta. 104 +- 13.57
Scale 10C m /cm

Figure 5. Cross Section 4 Antilao River

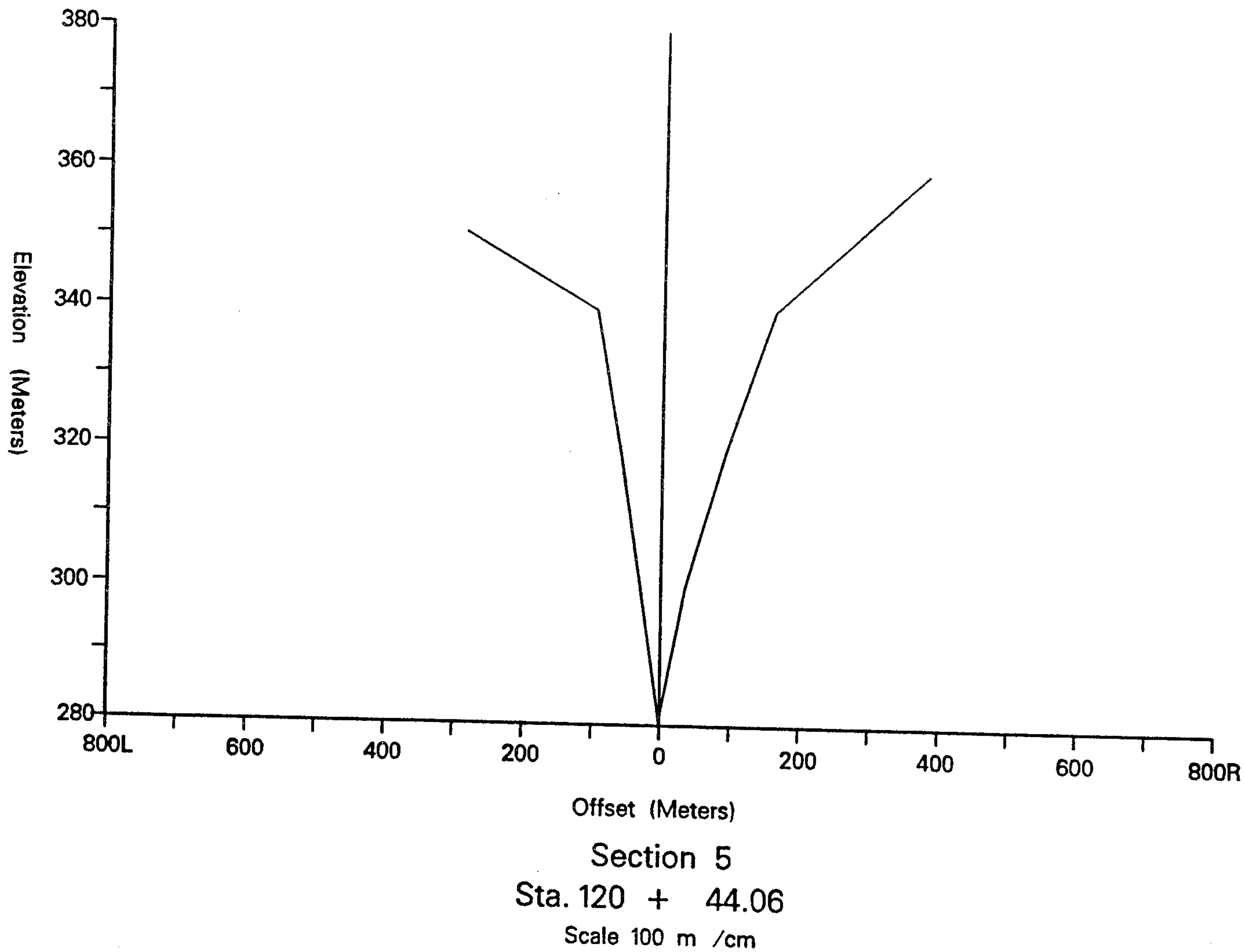


Figure 6. Cross Section 5 Antilao River

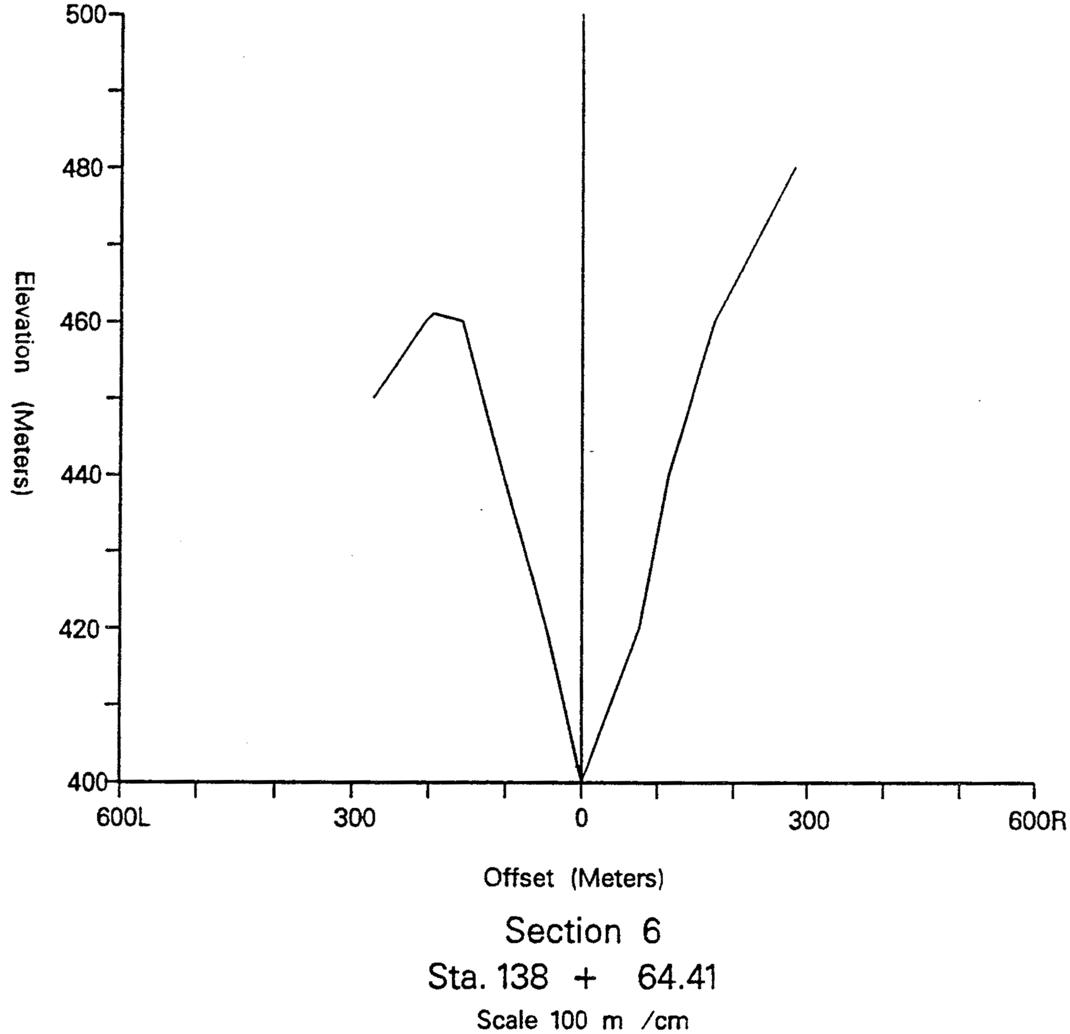


Figure 7. Cross Section 6 Antilao River

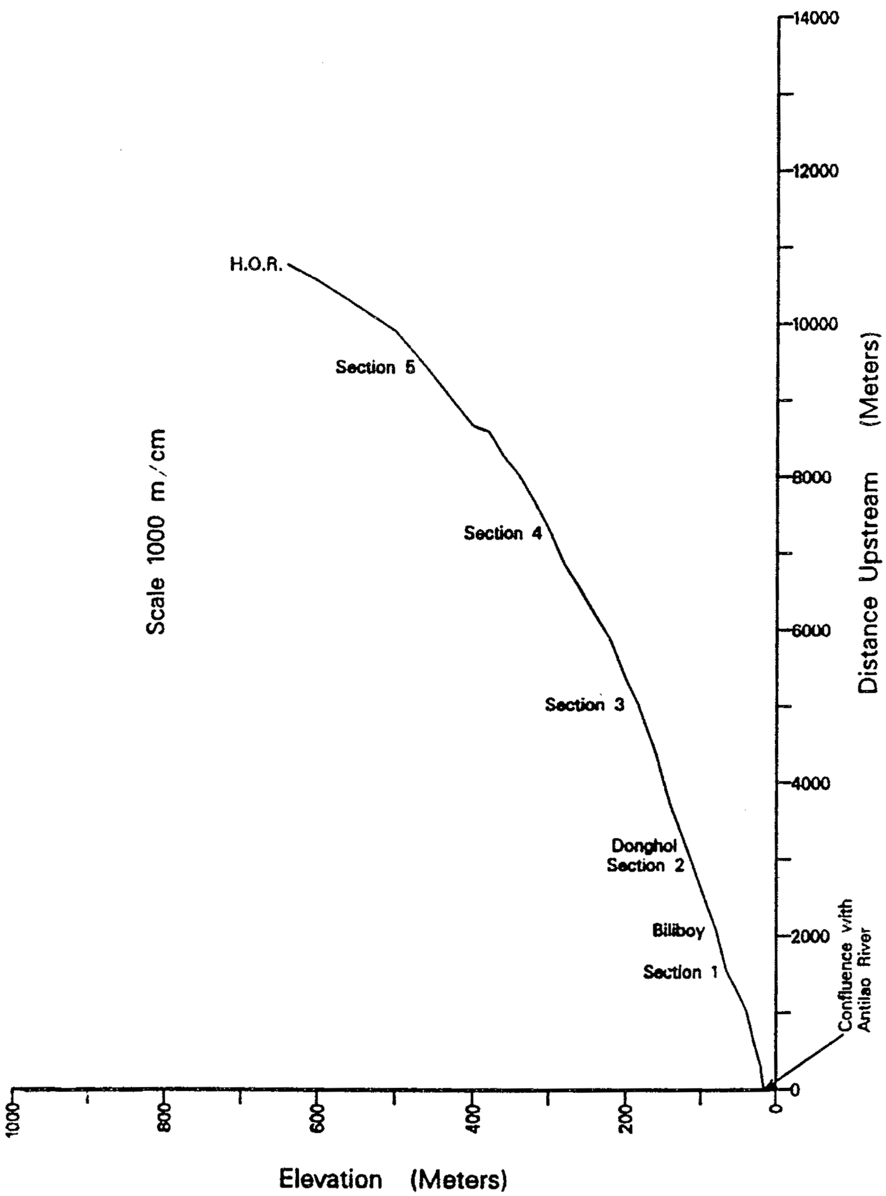


Figure 8. Malbasag River Channel Profile and Cross-Section Stationing

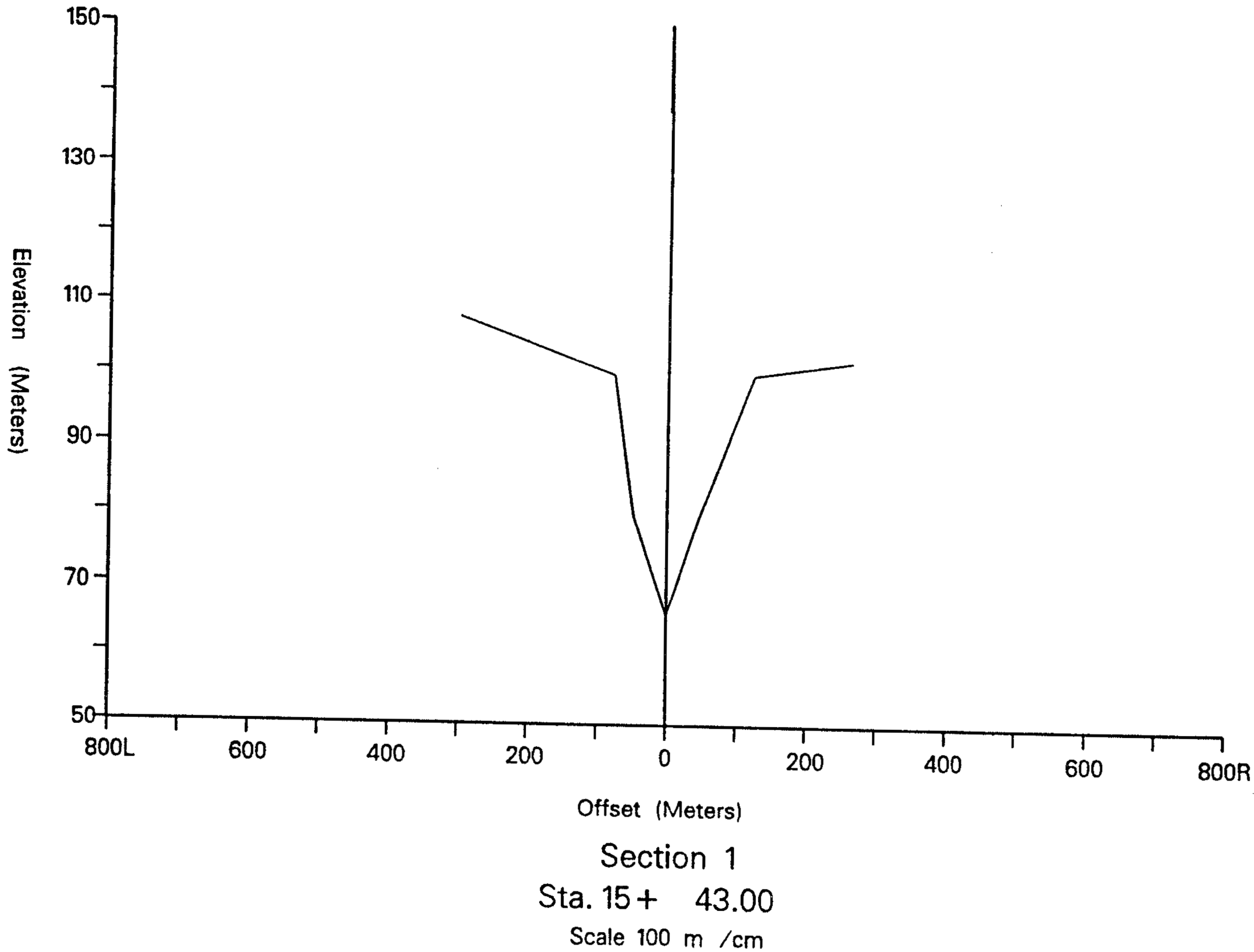


Figure 9. Cross Section 1 Malbasag River

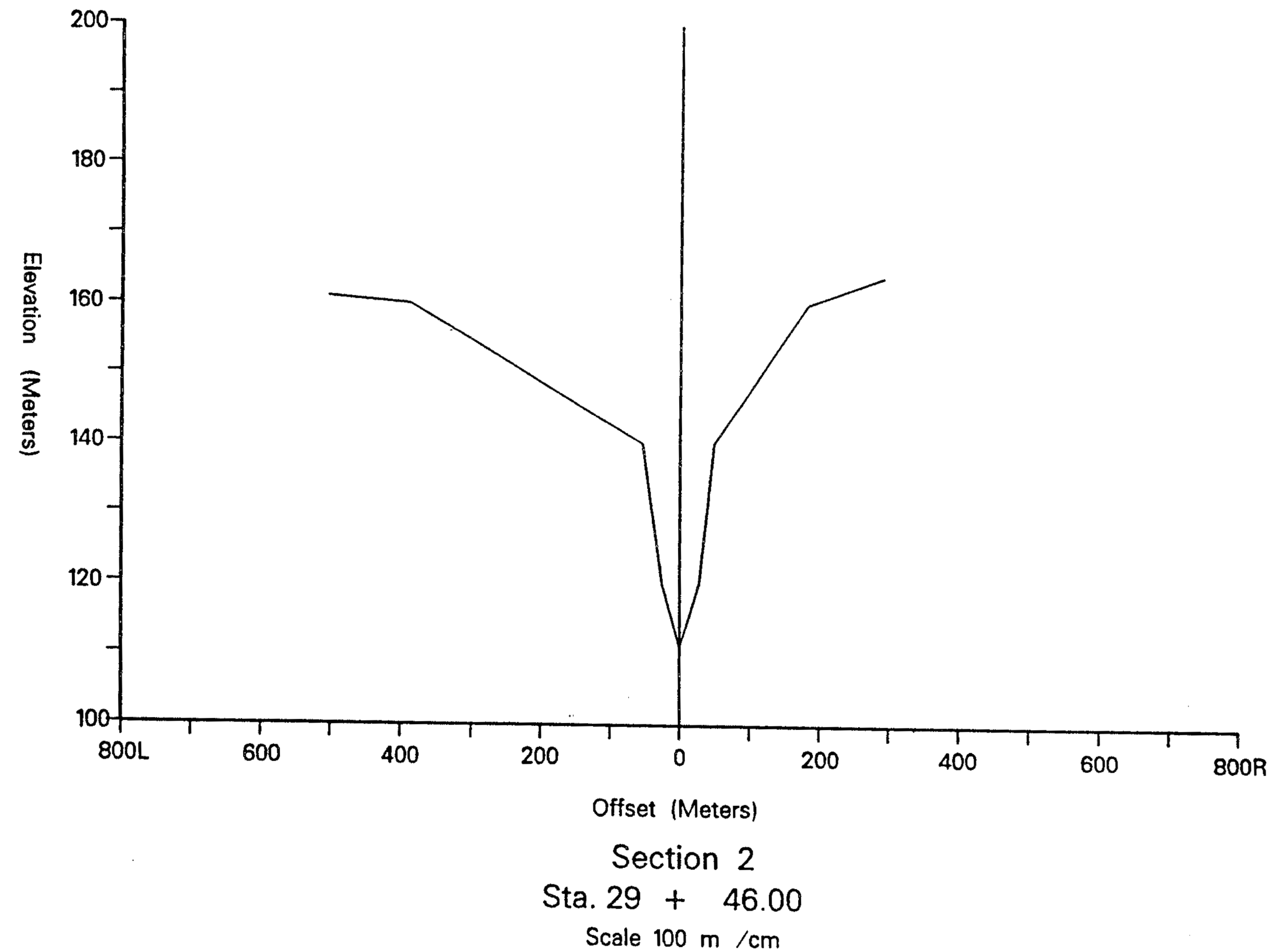


Figure 10. Cross Section 2 Malbasag River

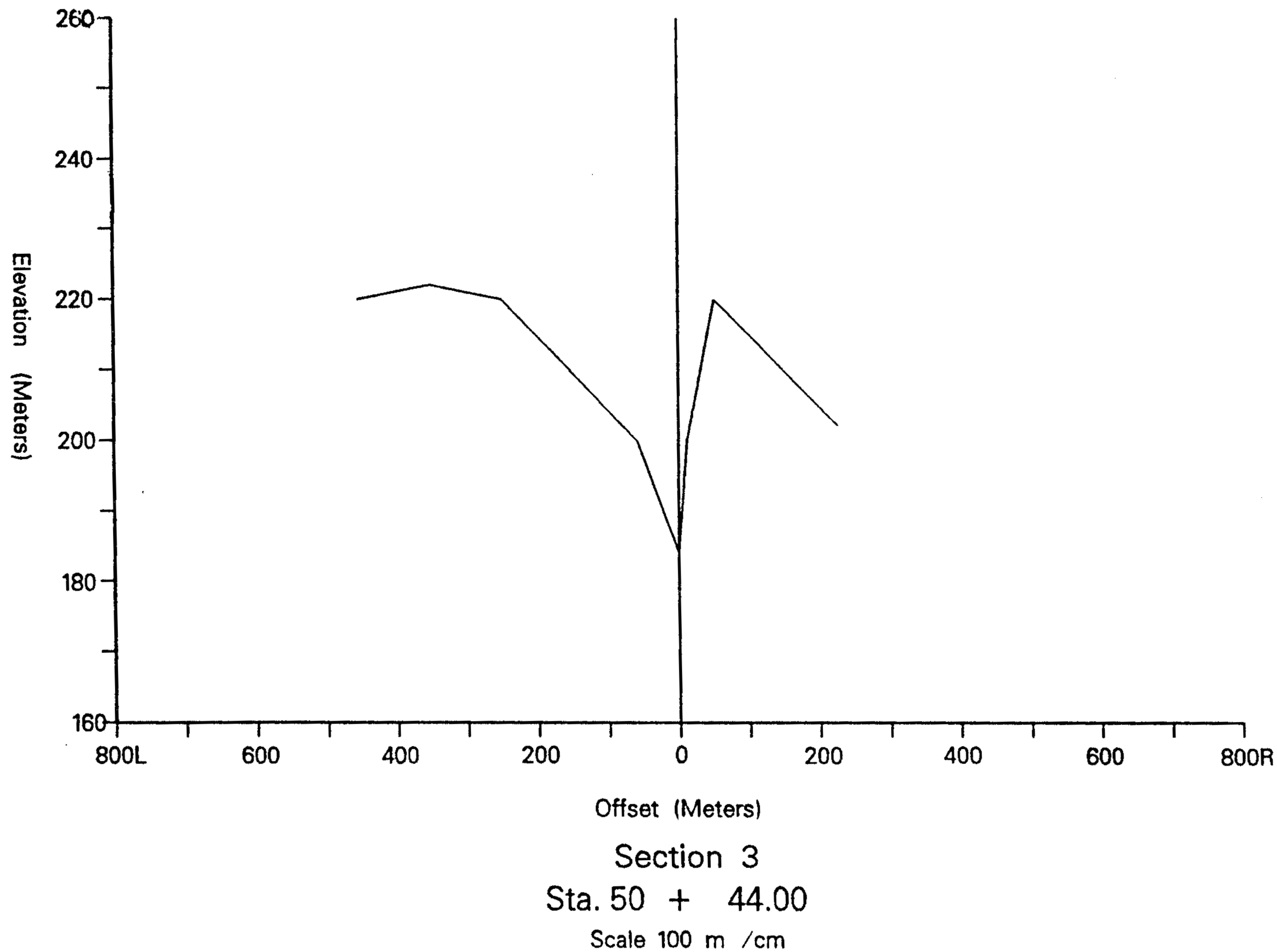


Figure 11. Cross Section 3 Malbasag River

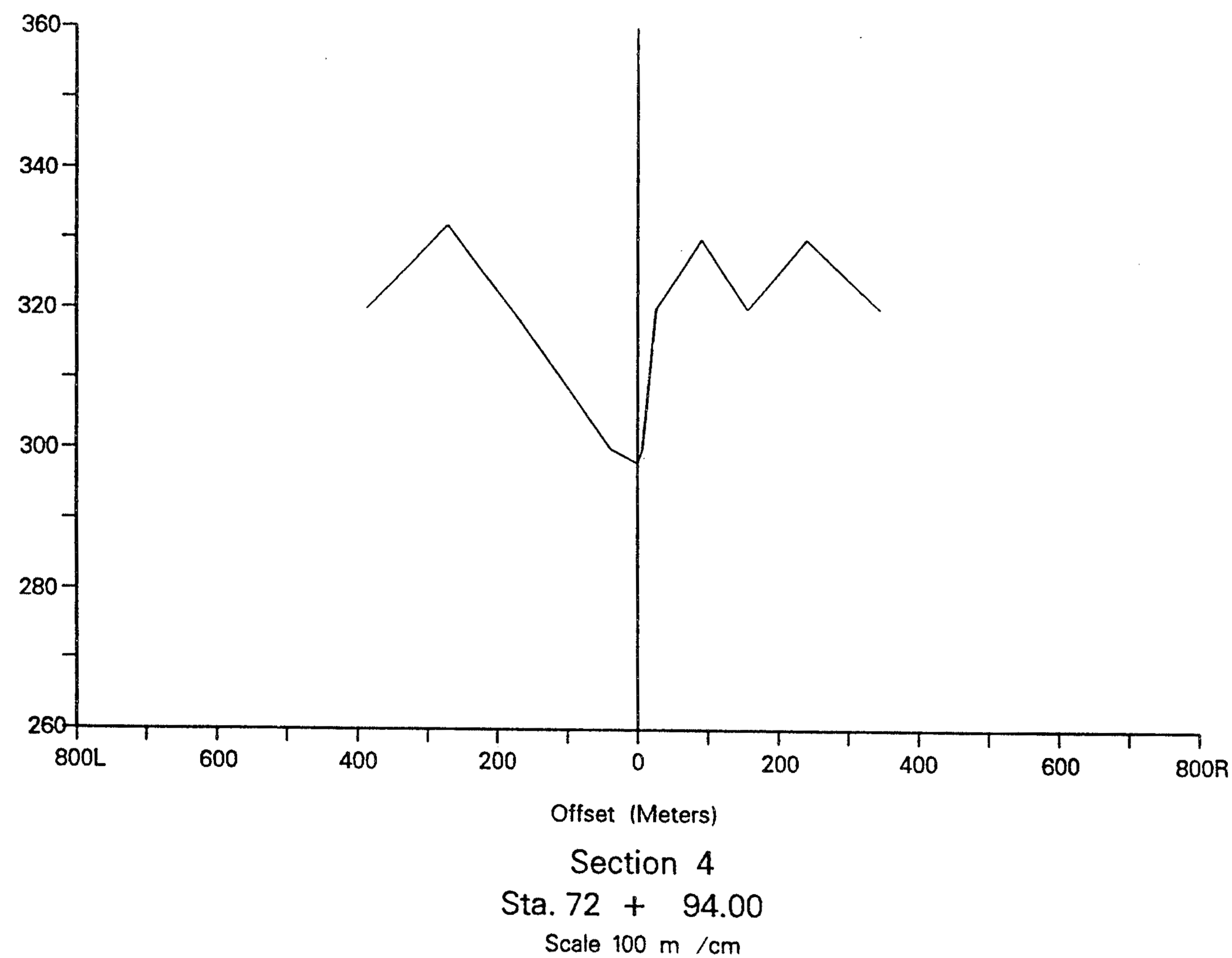


Figure 12. Cross Section 4 Malbasag River

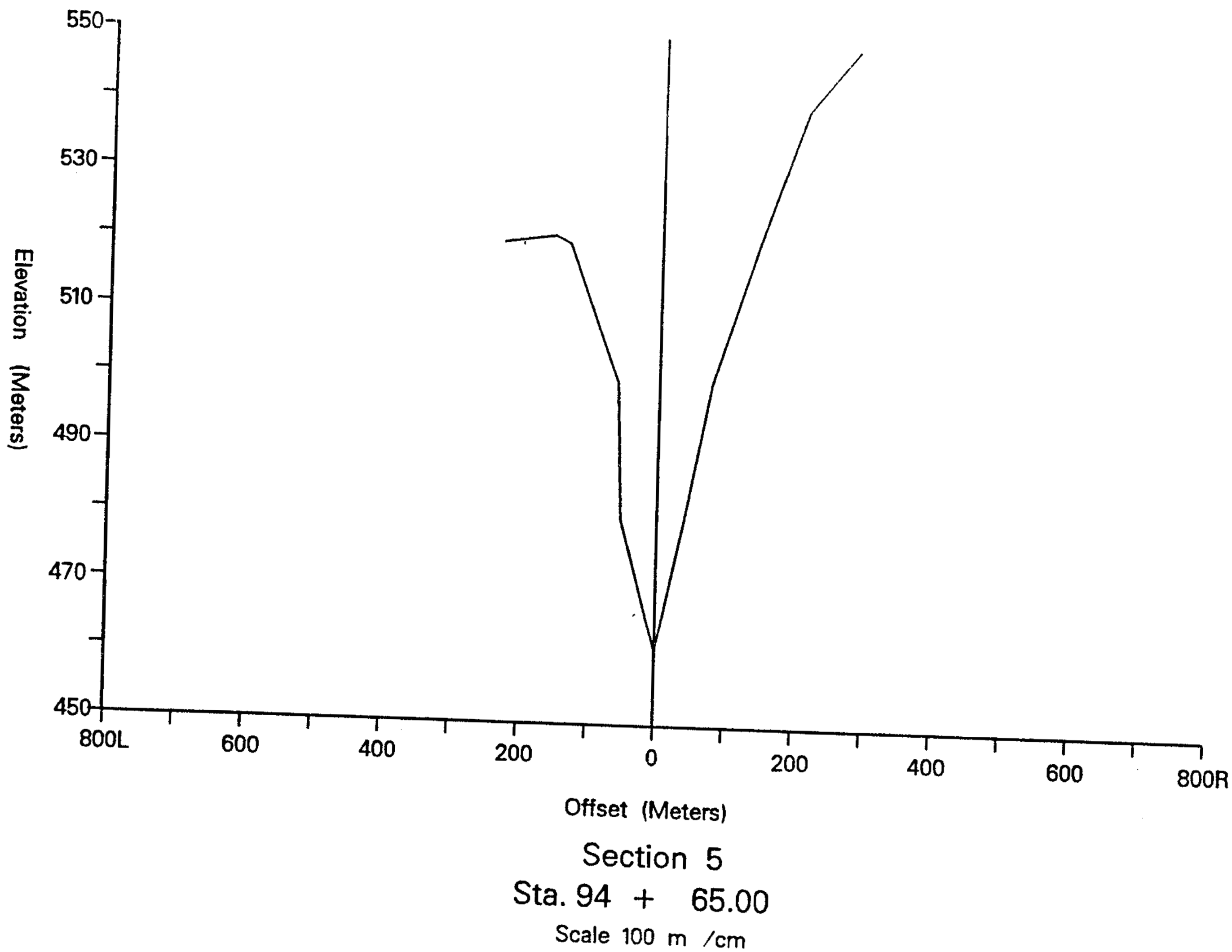


Figure 13. Cross Section 5 Malbasag River

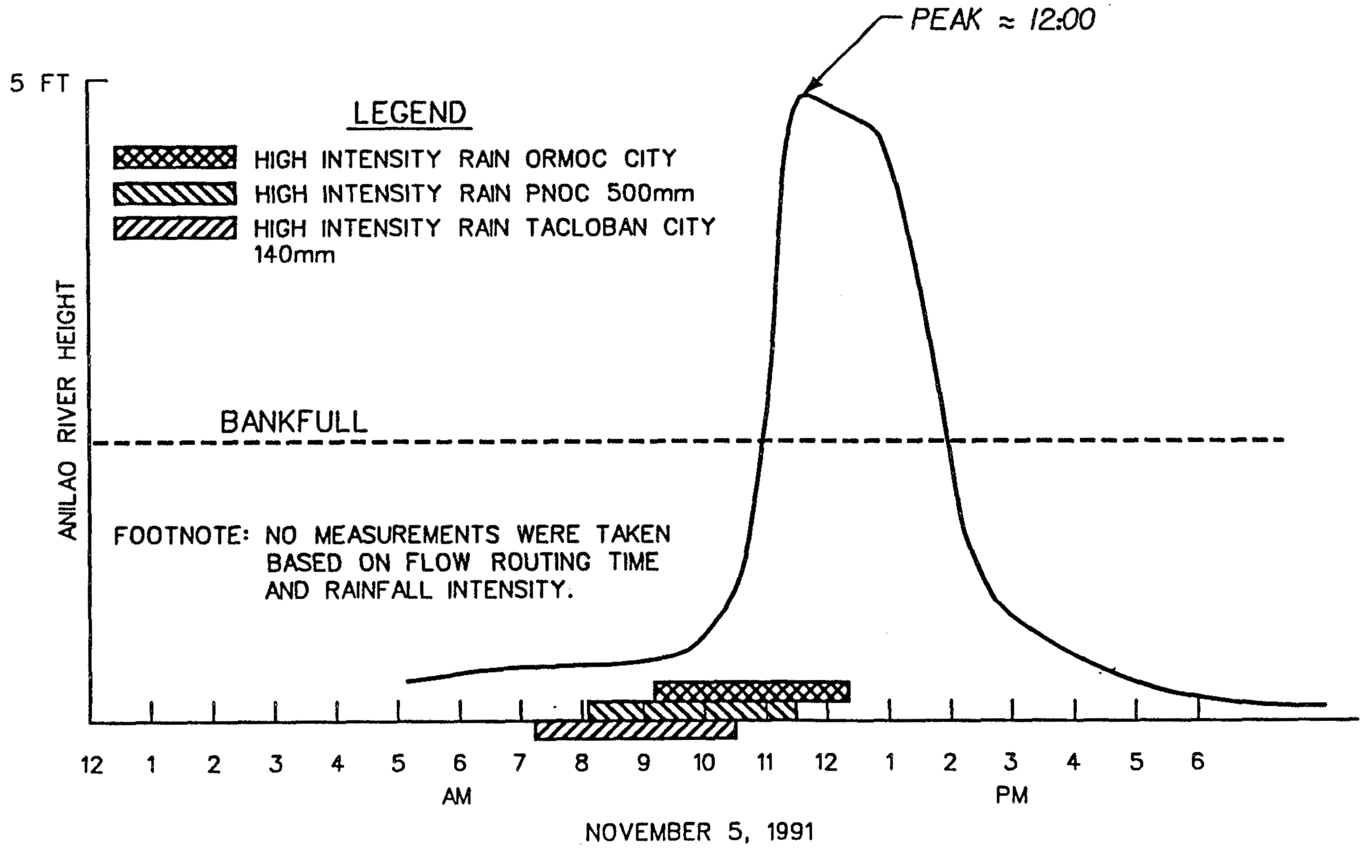


Figure 14. Antilao River Stage Hydrograph at Ormoc City

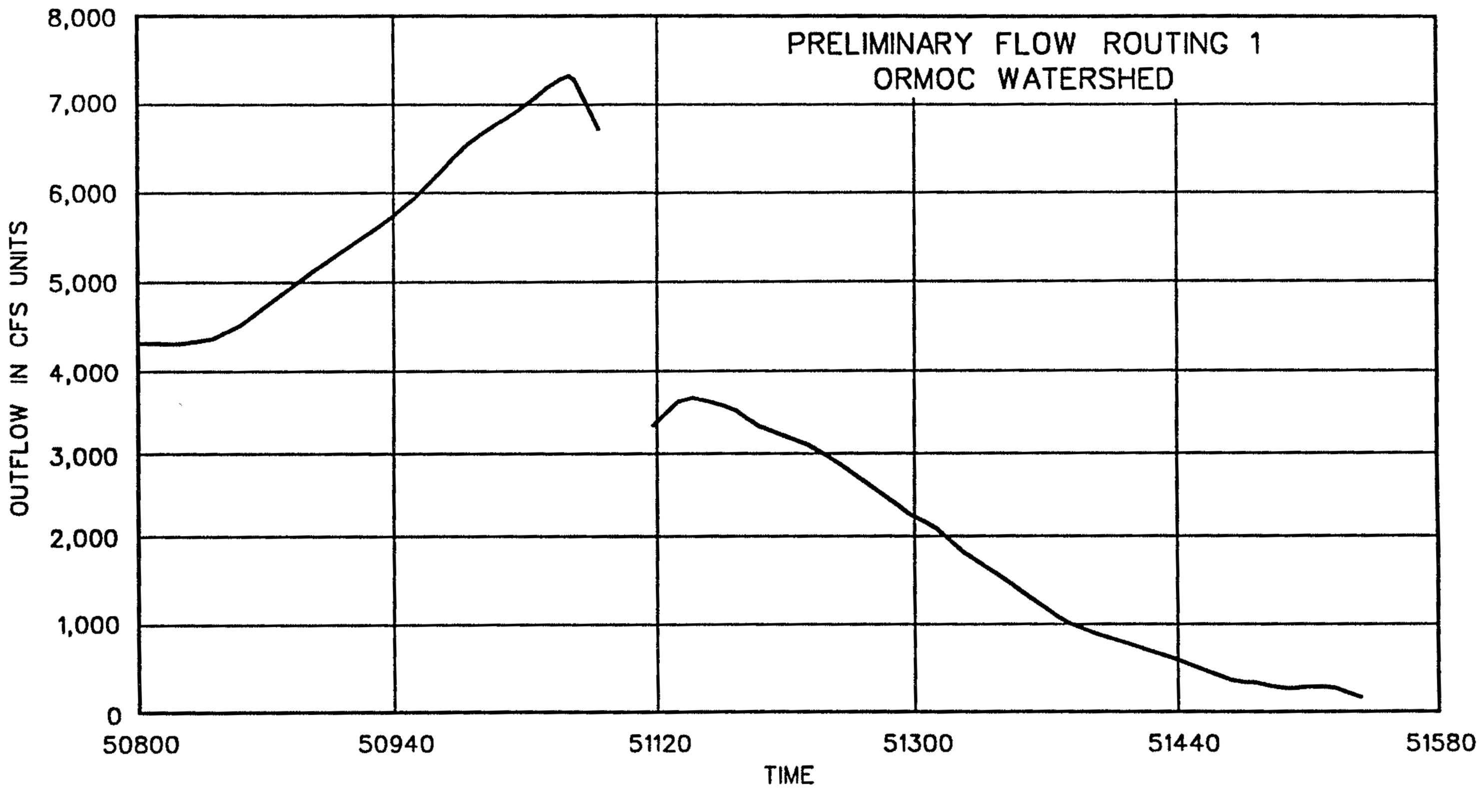


Figure 15a. Flood Routing #1 for Ormoc Watershed

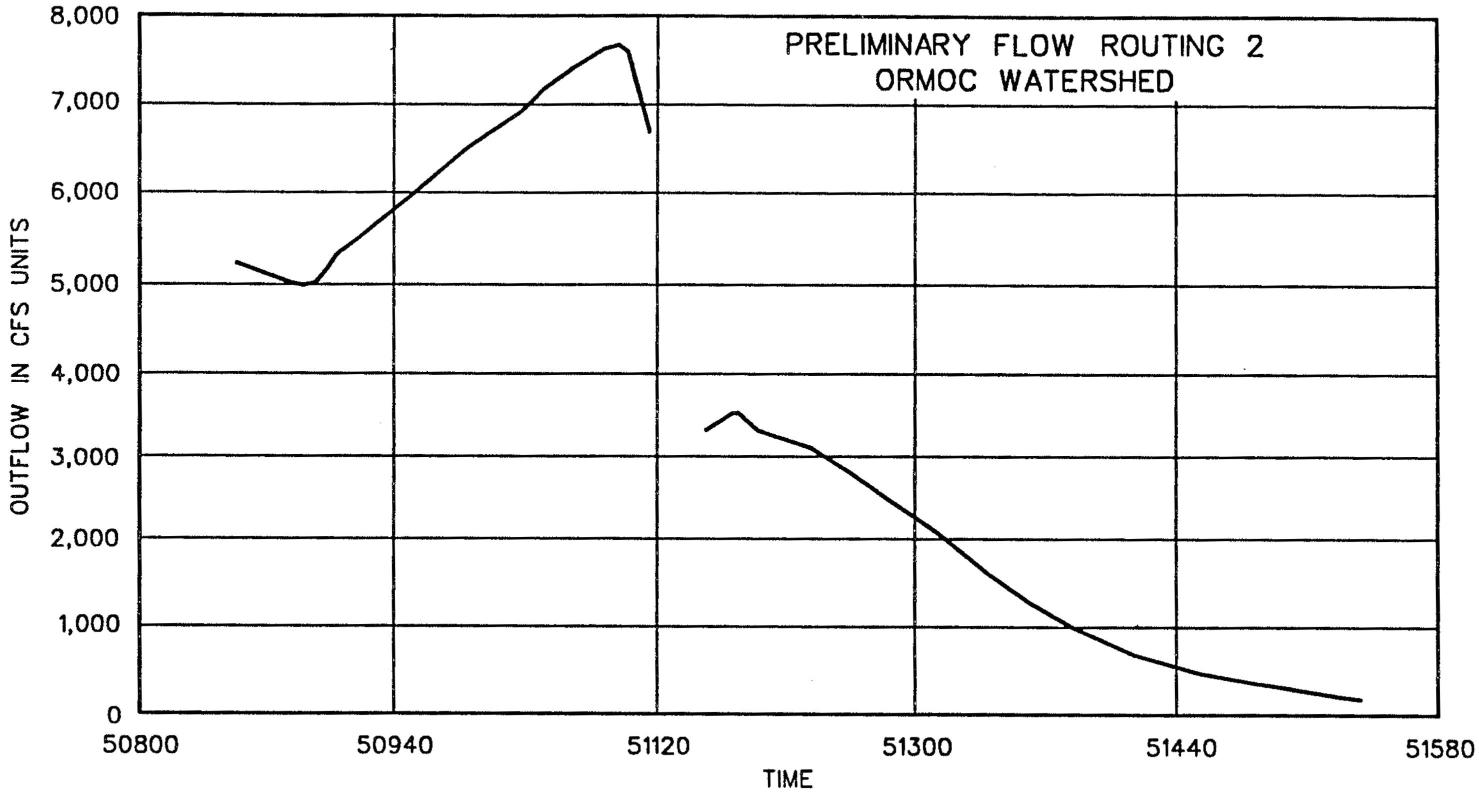
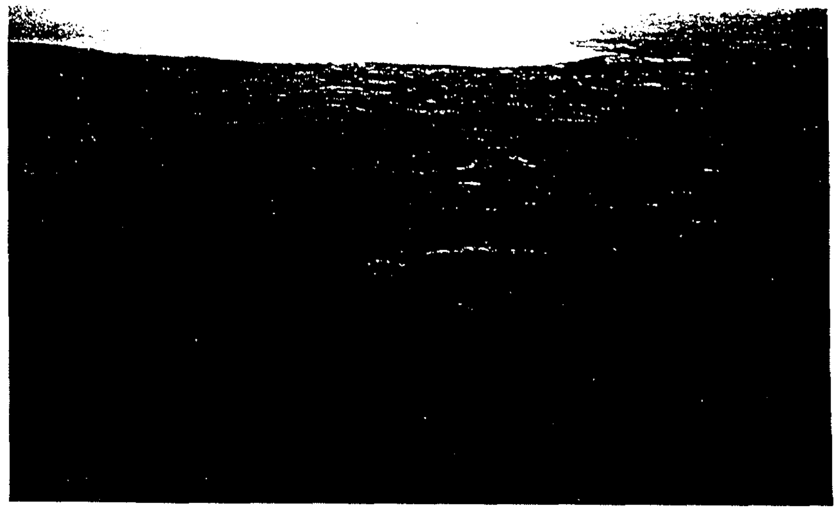


Figure 15b. Flood Routing #2 for Ormoc Watershed

a. UPPER DRAINAGE BASIN ORMOC WATERSHED
ANTILAO RIVER

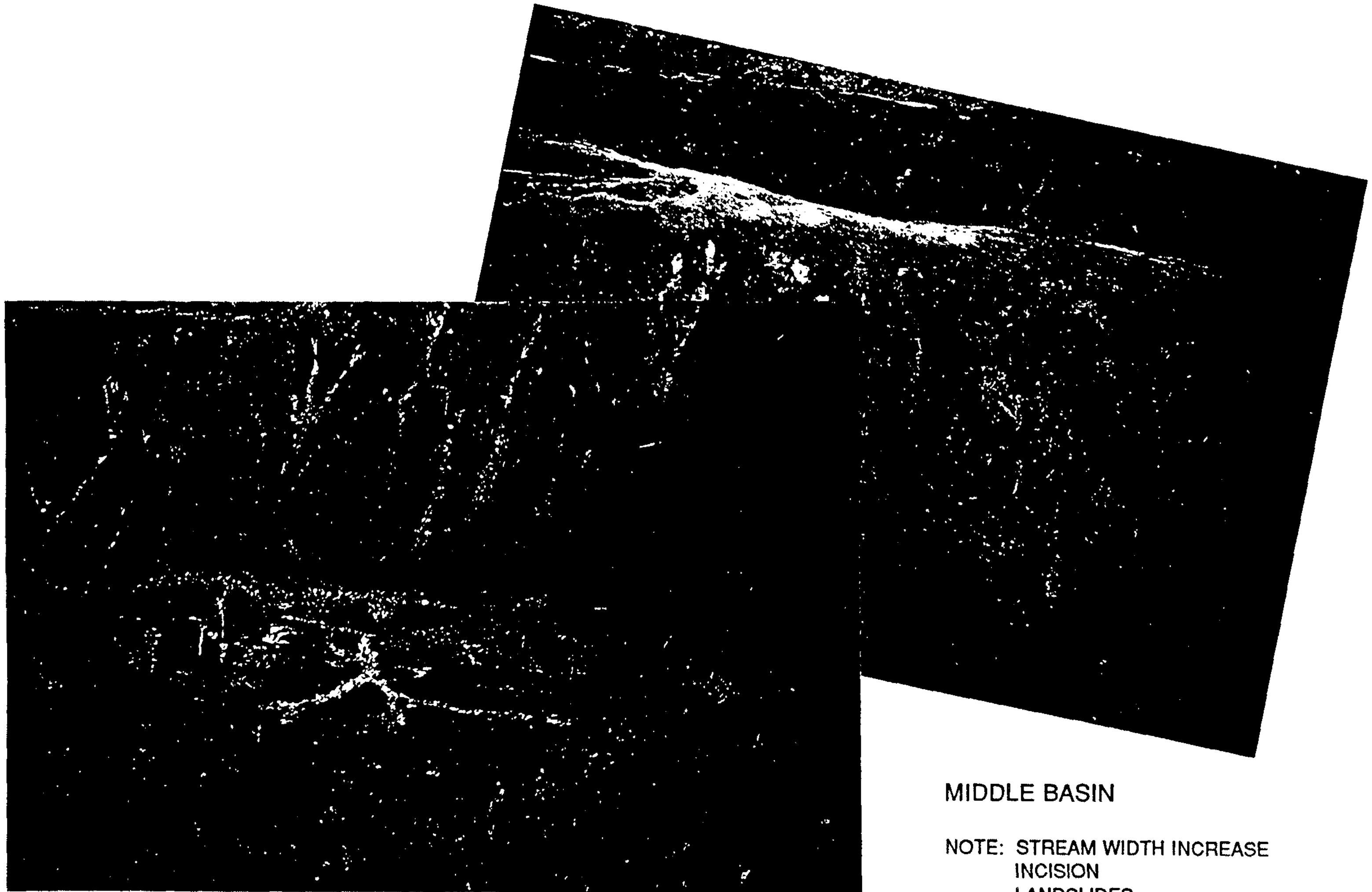
NOTE: CHANNEL INCISION
LANDSLIDE DEPTH AND LENGTH
STREAM SIDE SLOPES



b. LOOKING DOWN SYSTEM TO ORMOC CITY

NOTE: DENDRITIC CHANNEL PATTERN

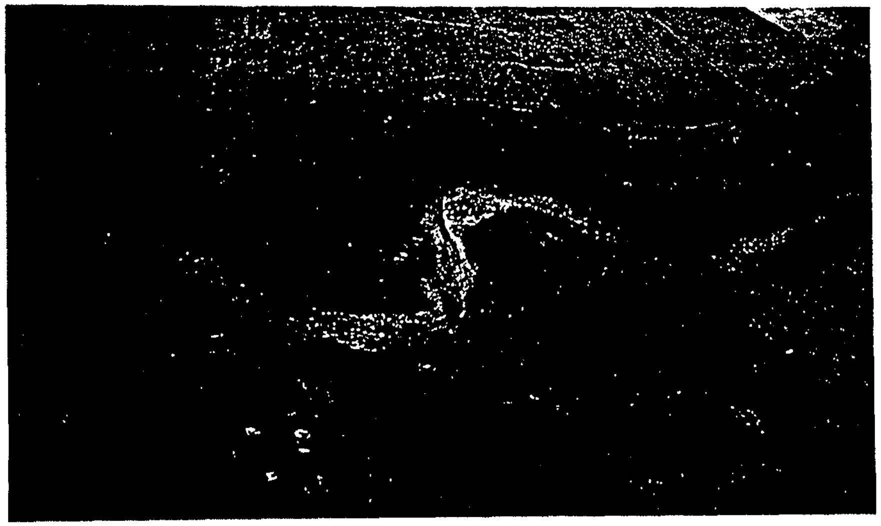
PHOTO PLATE 1



MIDDLE BASIN

NOTE: STREAM WIDTH INCREASE
INCISION
LANDSLIDES
STREAM SIDE SLOPES

PHOTO PLATE 2



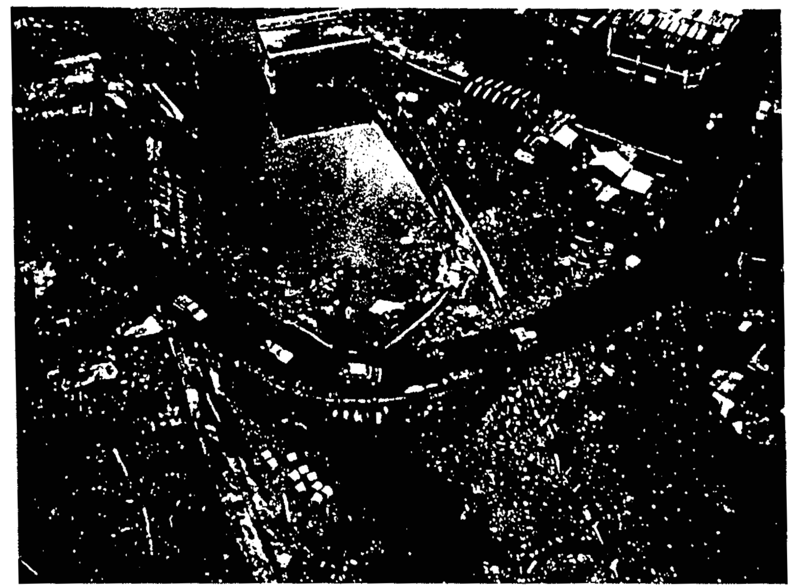
LOWER BASIN

NOTE: CHANNEL MEANDERS
INCISION
DECREASE IN CHANNEL SIDE SLOPE
REDUCTION IN LAND SLIDE OCCURRENCE

PHOTO PLATE 3

a. WOODY DEBRIS DEPOSITION AREA UPSTREAM OF ANTILAO RIVER BRIDGE

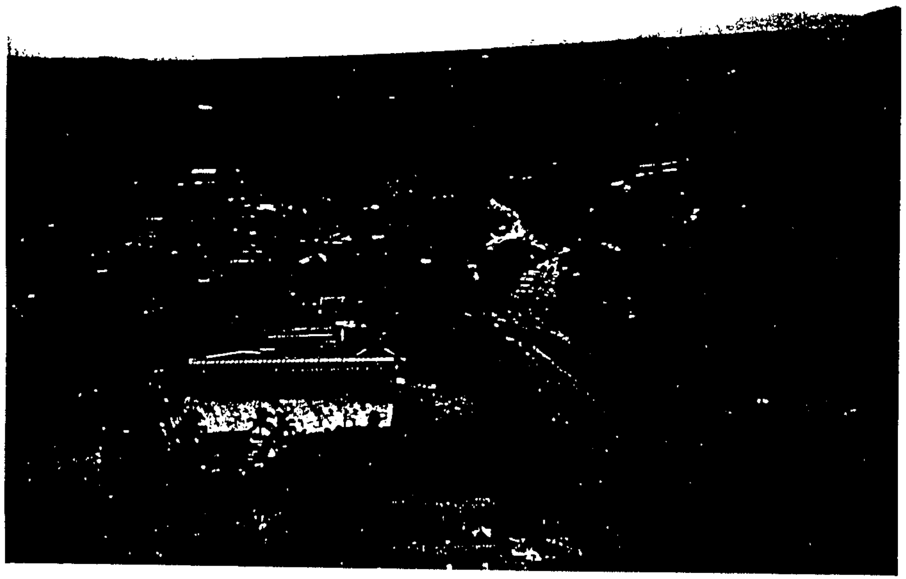
NOTE: 1. CHANNEL MAKES 90° BEND AT THIS LOCATION
2. FLOW MOMENTUM DIRECTED TO RIGHT OF PHOTO WHICH IS PROBABLE 1ST OVERFLOW ZONE INTO CITY



b. TYPICAL CHANNELIZED SECTION THROUGH ORMOC CITY

NOTE: FAILED RIVER BRIDGE

PHOTO PLATE 4



a. ANTILAO RIVER UPSTREAM OF ISLA VERDE AREA
ANTILAO RIVER AND MALBASAG RIVER
CONFLUENCE AREA

b. FLOOD PLAIN HABITATION AREA
U/S OF ORMOC BRIDGE AND
HIGHWAY 302 TRAINING OF
ANTILAO RIVER, MALBASAG RIVER

PHOTO PLATE 5

a. FLOOD PLAIN REHABITATION
ISLA VERDE AREA

b. ANTILAO BRIDGE AT ORMOC

NOTE: CHANNELIZATION DOWNSTREAM OF FAILED
BRIDGE STRUCTURE 90° CHANNEL BEND
DEBRIS DEPOSITION ZONE
*DDZ = DEBRIS DEPOSITION ZONE

PHOTO PLATE 6

TREE BLOW DOWN UPPER ORMOC WATERSHED

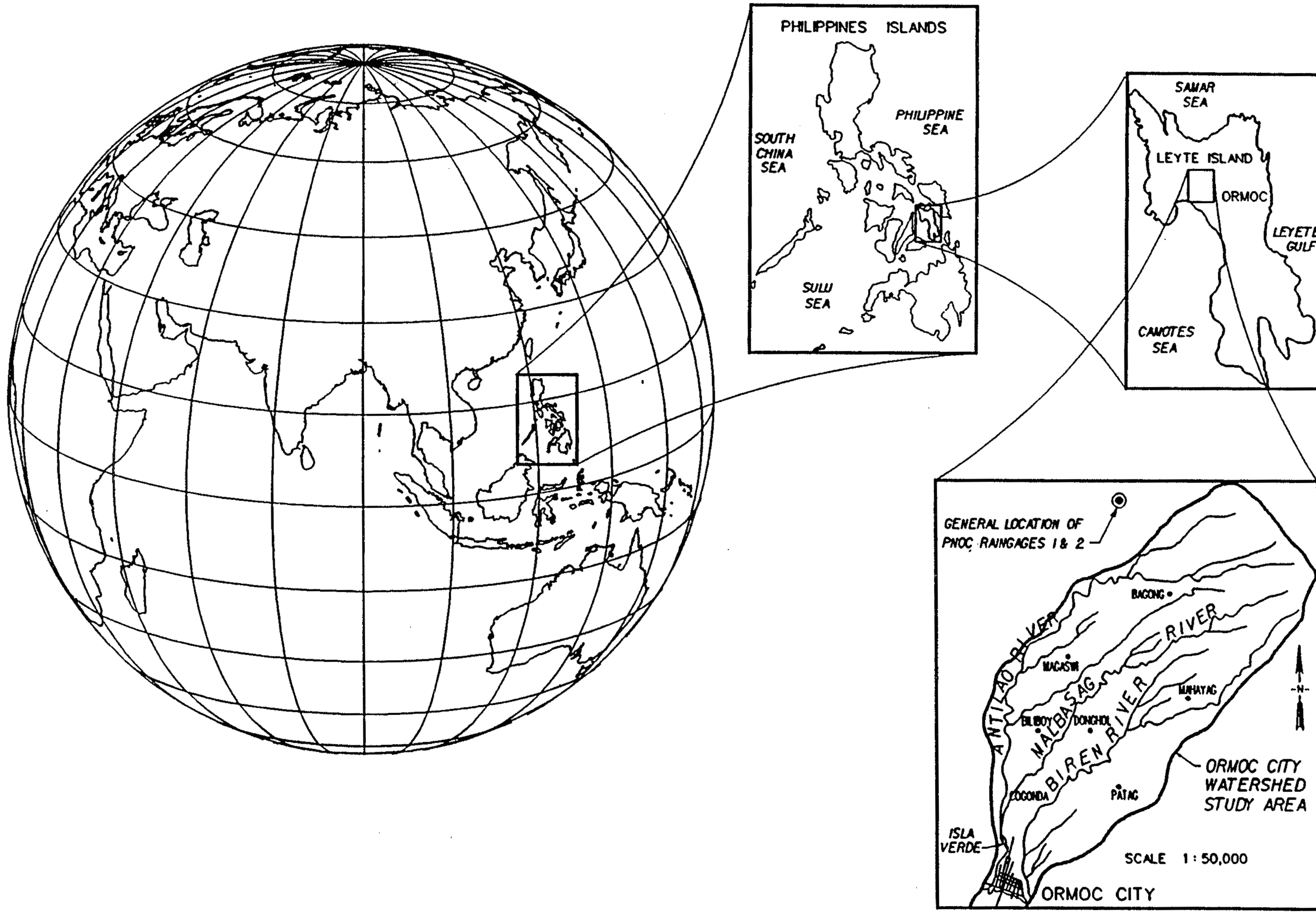
NOTE: LARGE SHALLOW MASS FAILURES

PHOTO PLATE 7

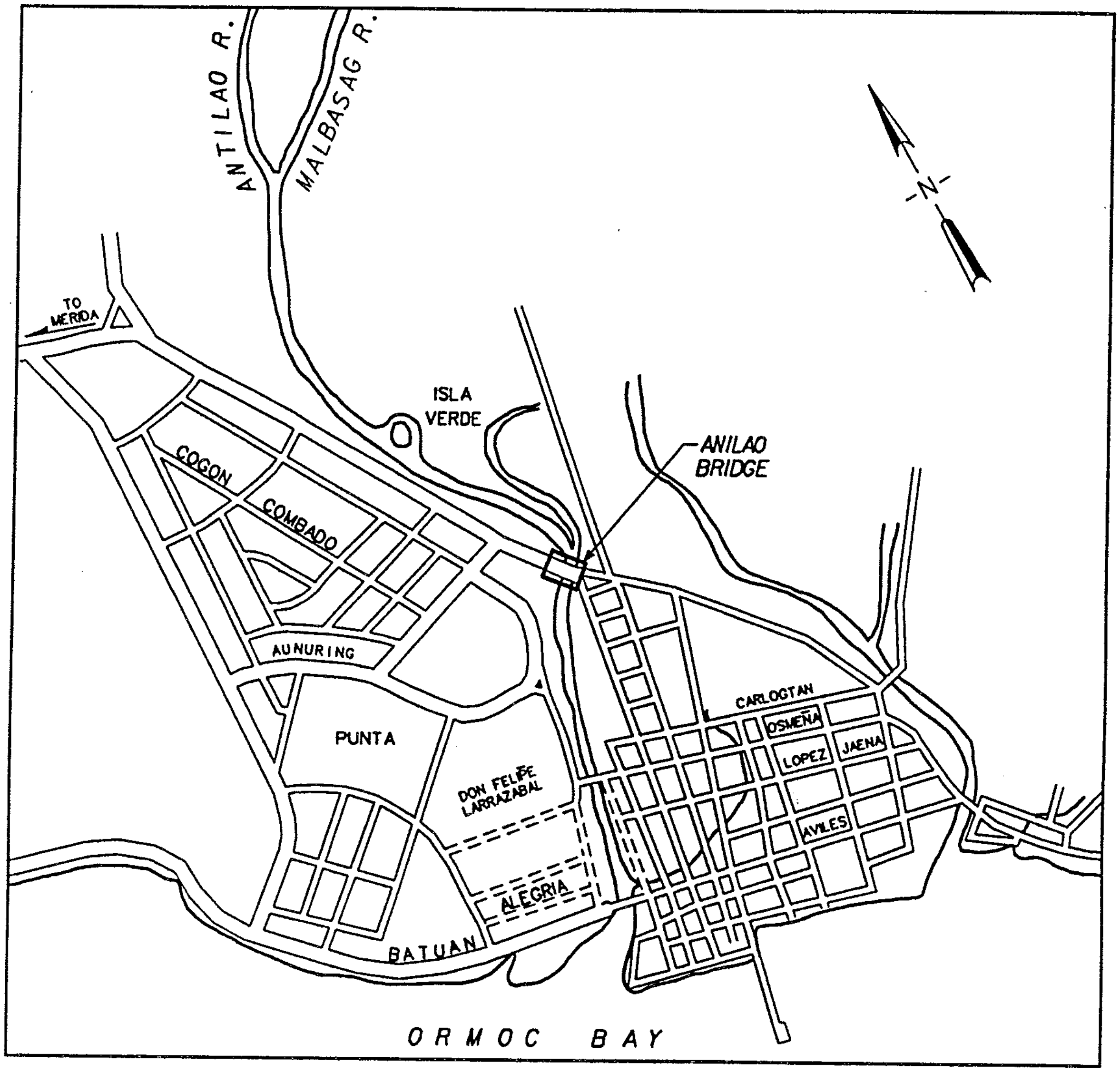
a. BRIDGE FAILURE PALANAS BRIDGE

b. BRIDGE FAILURE CALGIGA-A BRIDGE

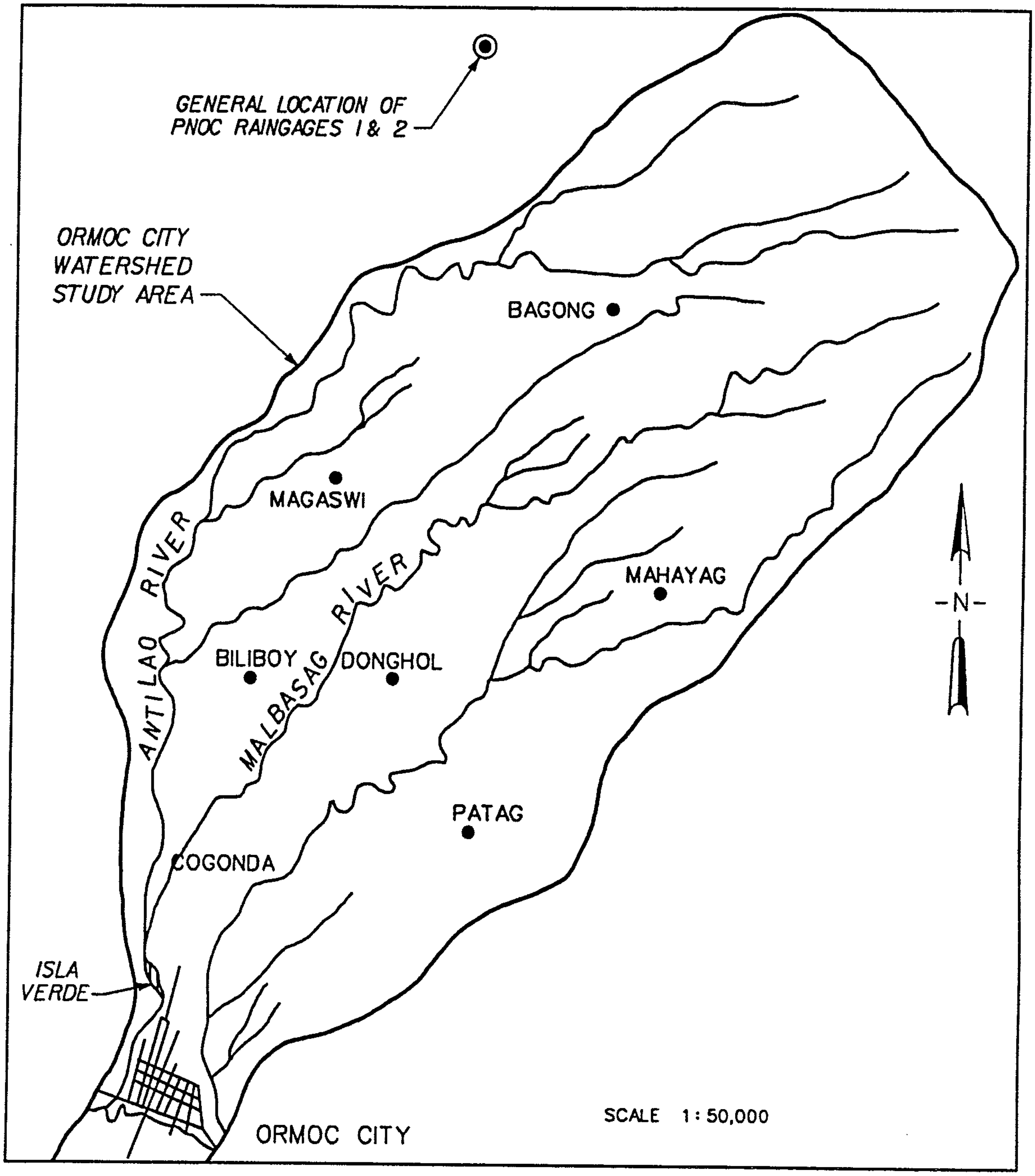
PHOTO PLATE 8



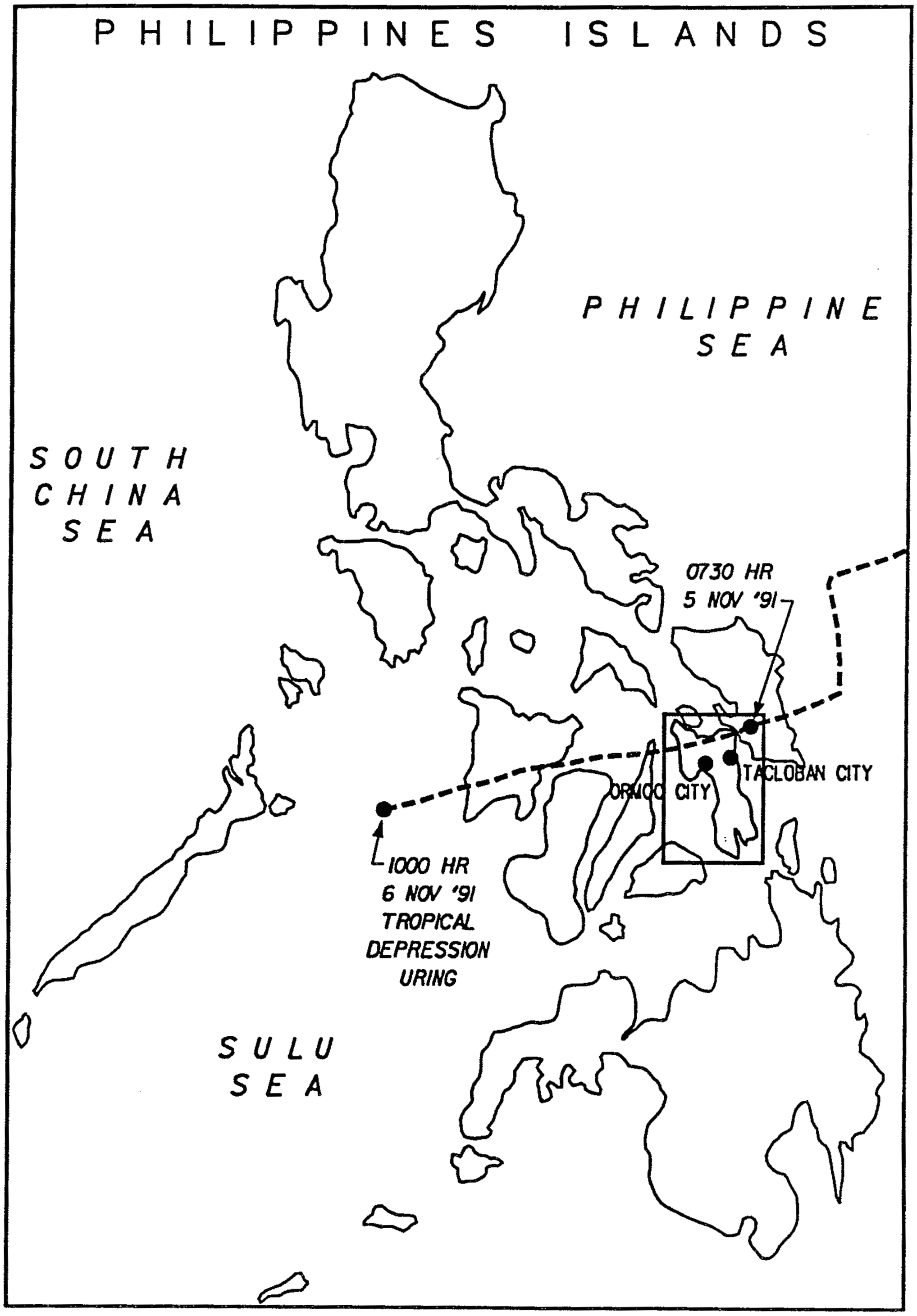
Map 1. Location Map, Ormoc Watershed, The Phillippines



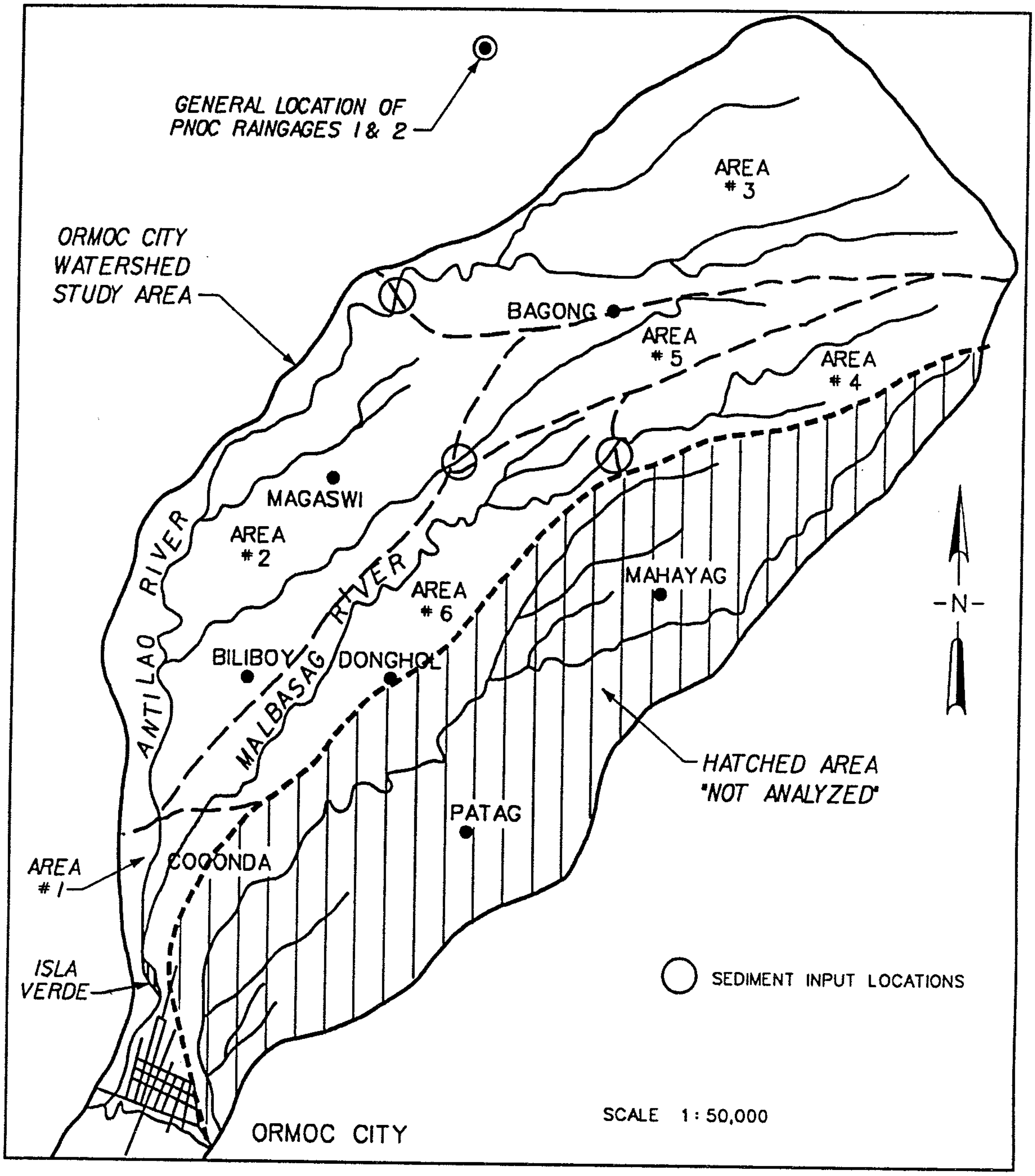
Map 2. Ormoc City Area Map, Two Rivers and Flooded Areas



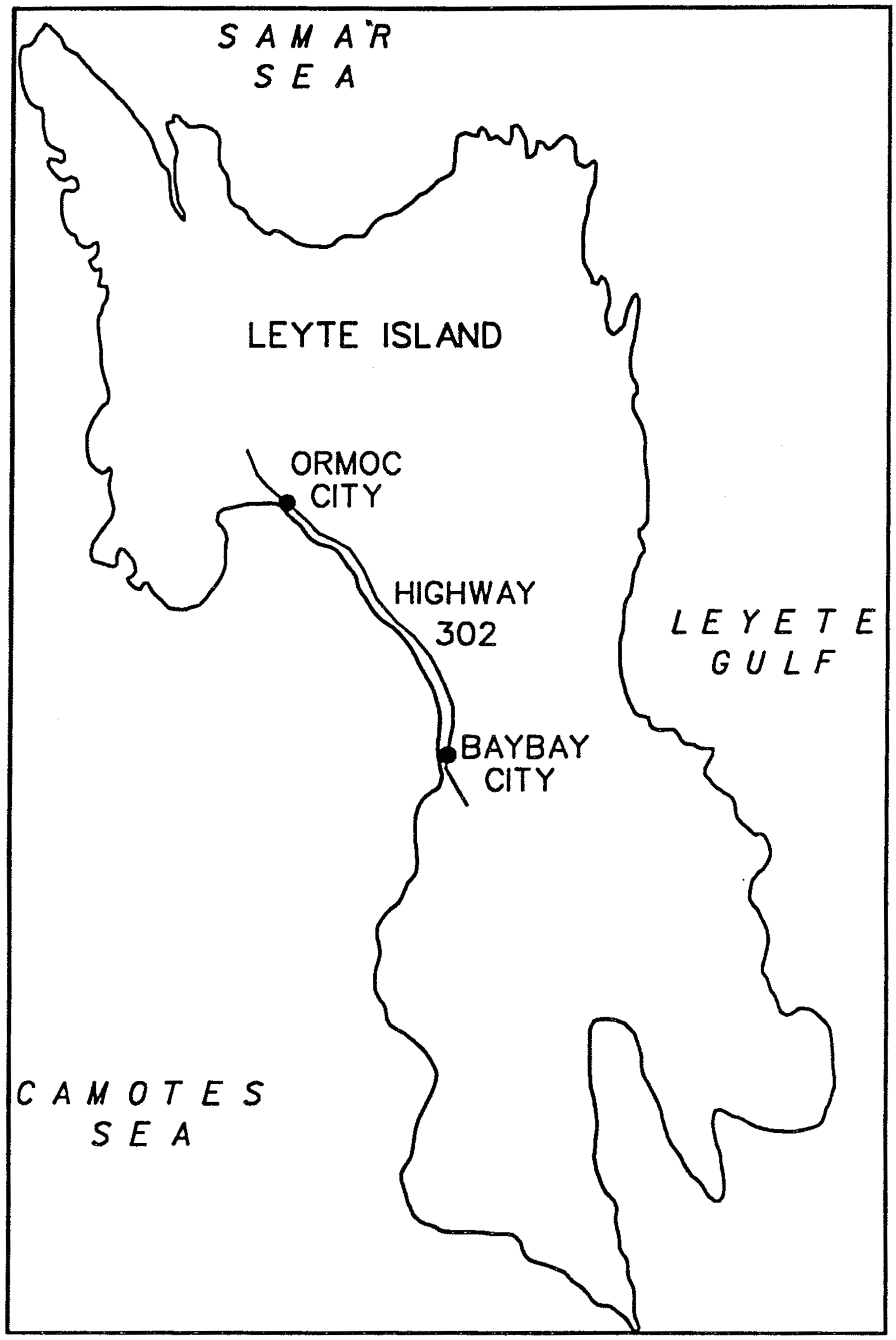
Map 3. Ormoc Watershed



Map 4. Storm Track for Typhoon URING



Map 5. Sediment Routing Basin Map



Map 6. Ormoc City to Baybay Leyte
Coastal Highway No. 302

Appendix A: List of Contacts

Vicente Paragas	Regional Technical Director, Department of Environment and Natural Resources
Rosalio Goze	Regional Executive Director, Department of Environment and Natural Resources
John Sturnes	U.S. AID Office of Capital Projects, U.S. State Department
Bob Vergara	U.S. AID Office of Capital Projects, U.S. State Department
Abelardo M. Monge, Jr.	Assistant Regional Director for Services, Department of Public Works and Highways
Pacifico G. Mendoza, Jr.	Regional Director, Department of Public Works and Highways
Eriverto V. Loreto	Congressman, 5th District, Leyte
Carment L. Cari	Mayor, Baybay, Leyte
Ruben Penserga	City Engineer, Ormoc, Leyte
Ramon Omega	District Engineer, Leyte 2, Department of Public Works and Highways
Leonardo A. Nunez	Director, Bureau of Maintenance, Department of Public Works and Highways
Fortunato Dejoras	Administrator, Office of Civil Defense
Romulo M. del Rosario	Undersecretary, Bureau of Maintenance, Department of Public Works and Highways
Vicente Yulo	Third Leyte Engineering District, Assistant District Engineer
Minni Dilgo	Philippine National Oil Company, Assistant Service Manager
Janparralla	Weather Data

ENGINEERING OFFICER
USAID/Philippines

Date: 05 December 1991

To: File

Subject: ORMOC DISASTER
Corps of Engineers

Revised agenda for evaluation team:

05 DEC 91	2250 hrs	Team arrives NAIA
05 DEC 91	2320 hrs	Team departs NAIA for Sheraton
05 DEC 91	2350 hrs	Team arrives at Sheraton
06 DEC 91	0845 hrs	USAID vehicle picks up Team at Sheraton for travel to RMC
06 DEC 91	0900 hrs	Team arrives at RMC and meets with USAID technical staff
06 DEC 91	1100 hrs	Team meets with USAID Director
06 DEC 91	1330 hrs	Team departs RMC for DPWH
06 DEC 91	1400 hrs	Team meets with DPWH Undersecretary
06 DEC 91	1530 hrs	Team departs DPWH for Sheraton
06 DEC 91	1600 hrs	Team arrives at Sheraton
08 DEC 91	1430 hrs	USAID vehicle picks up Team at Sheraton for travel to Manila domestic airport
08 DEC 91	1500 hrs	Team arrives at Manila domestic airport
08 DEC 91	1600 hrs	Team departs Manila via PAL
08 DEC 91	1705 hrs	Team arrives Tacloban
09 DEC 91	0830 hrs	Team departs Tacloban for Ormoc via GOP helicopter
09 DEC 91	1600 hrs	Team departs Ormoc for Tacloban via GOP helicopter
10 DEC 91	0830 hrs	Team departs Tacloban for Ormoc via GOP helicopter
10 DEC 91	1500 hrs	Team departs Ormoc for Tacloban via GOP helicopter
10 DEC 91	1805 hrs	Team departs Tacloban via PAL
10 DEC 91	1940 hrs	Team arrives Manila
11 DEC 91		Team drafts report and prepares for exit briefing
12 DEC 91		Team drafts report and prepares for exit briefing
13 DEC 91	0800 hrs	USAID vehicle picks up Team at Sheraton for travel to RMC
13 DEC 91	0815 hrs	Team arrives at RMC and meets with USAID technical staff
13 DEC 91	0830 hrs	Team gives exit briefing at USAID
13 DEC 91	1030 hrs	Team departs RMC for DPWH

U.S. AGENCY FOR INTERNATIONAL DEVELOPMENT

Ramon Magsaysay Center
1680 Roxas Boulevard
Ermita 1000, Manila
Philippines

Fax No.: 632-521-5241
Tel. No.: 632-521-7116

FAX TO: **RAY DIONNE**
OFDA/AID/W

FAX NO: (202) 647-5269

This is a formal request for the River Morphologist. I have sent you by separate fax some comments to help you in working out his contract and costs. This is the formal request. We have the attached letter from the GOP asking for this help. The report will be a report to the GOP and the USG. The report for the GOP to go to the Department of Public Works and Highways and the USG to OCP.

Bryant George
M D R O
11/20/91

Attachment: a/s

wp/bgdoc./11/20/91

REPUBLIC OF THE PHILIPPINES
DEPARTMENT OF PUBLIC WORKS AND HIGHWAYS
OFFICE OF THE SECRETARY
MANILA

RECEIVED
NOV 20 1 58 PM '91
110718
USAID/C&R

November 19, 1991

MR. MALCOLM BUTLER
Director
United States Agency for
International Development
Manila, PHILIPPINES

LP
MM

ATTENTION: MR. LEROY PURIFOY
Chief Engineer, OCP

REFERENCE: TYPHOON URING DAMAGE ASSESSMENT

SUBJECT: REQUEST FOR THE ENGAGEMENT OF A SHORT TERM DISASTER EVALUATION TEAM BY USAID

Dear Mr. Butler:

The Department of Public Works and Highways is undertaking an investigation to determine the cause of the flooding which devastated Ormoc and the surrounding communities following Typhoon Uring in early November 1991. We therefore seek the assistance of USAID in providing a team of disaster evaluation specialists from the US Army Corps of Engineers who will help the Department in the evaluation of the damage and in providing insight to the cause and possible mitigation measures for the disaster.

We hope that USAID will consider this request favorably and assign the specialist team as soon as possible. DPWH is grateful to USAID for their continued assistance and cooperation in our efforts to rehabilitate disaster areas in the Philippines.

Sincerely,

TEODORO T. ENCARNACION
Undersecretary

ACTION TAKEN

DUE DATE

U.S. AGENCY FOR INTERNATIONAL DEVELOPMENT

Ramon Magsaysay Center
1680 Roxas Boulevard
Ermita 1000, Manila
Philippines

Fax No.: 632-521-5241
Tel. No.: 632-521-7116

November 20, 1991

Mr. Ray Dionne
Office of Foreign Disaster
Assistance/ASP Room 1052 NS
AID Washington, D.C. 20523

Ray,

Thank you for forwarding the scope of work proposed by the River Morphologist. We have the following comments:

1. The base of operation needs to be Tacloban or Ormoc, very adequate accommodations are available. Manila is far too far away from this location to be of use as a base. This would be about an 4 hour a day commute.

2. We will attempt to get helicopter from private companies in the area or AFP. Again, its too far from the Subic Assets, the team we have down there now just notified me that they are taking a bus from one location to another. Then choppers did not materialize because of the newest storm Yayang, which makes it impossible to fly. Basically, we cannot guarantee that there will be helicopter support the whole time but we can guarantee that there will be some helicopter support. However, a great deal of this is going to have been done over muddy roads and none to secure bridges.

3. We can arrange for full meeting with and debriefing by the Dept. of Public Works and Highways (DPWH) people of the country and meeting with the DAST people who have made their evaluation of the area on November 15, 1991. In addition, we have excellent contacts with the shipping line people and transport people who handle all of the freight to and from Ormoc from Cebu City.

4. The per diem for Ormoc is $45.00 a day (lodging is $23.00 & meals are $22.00). The per diem for Tacloban is $57.00 a day (lodging is $39.00 & meals are $18.00).

5. We will make a formal request for the service of this man/men as soon as we get a formal request from the GOP. This is a highly sensitive issue and we must have a GOP Agency ready to handle the flack that surely will arise from any report that the USG develops no matter how it is vetted.

Best wishes,

Bryant George
Chief
Office of Food For Peace
and Voluntary Cooperatio

13 DEC 91	1100 hrs	Team arrives at DPWH and gives exit briefing
13 DEC 91	1230 hrs	Team departs DPWH

Bob Vargara, a USAID Foreign Service National engineer assigned to the region which includes Ormoc, and John Starnes, USAID Engineering Officer, will accompany the team to Ormoc. USAID/Manila does not have a fund cite for your local transportation; therefore, be prepared to pay approximately $100 (P2683.50) each for your roundtrip air fare (Manila-Tacloban-Manila) and claim reimbursement for same on your travel voucher.

Director Leonardo A. Nunez of the Bureau of Maintenance in the DPWH appears to be taking the lead on the GOP side and will accompany the team to Ormoc. Mr. Fortunato Dejoras (National Disaster Coordinating Council) will also accompany the team.

John C. Starnes
Office of Capital Projects

Appendix B: Rainfall Data NPOC

APPENDIX B

PHILIPPINE NATIONAL OIL COMPANY (PNOC) RAINGAGE 1 & 2 DATA, LEYTE

DAILY RAINFALL DATA (in mm/day)
For the month of November 1991

DATE	STATIONS		DAILY TOTAL	DAILY AVERAGE	REMARKS
	1	3			
1	Nil	Nil	–	–	
2	0.8	1.2	2.0	1.8	
3	4.5	2.1	6.6	3.3	
4	21.0	15.0	36.0	18.0	
5	580.5	350.0	930.5	465.2	Typhoon "Uring"
6	46.0	41.8	87.8	43.9	
7	Nil	Nil	–	–	
8	Nil	Nil	–	–	
9	Nil	Nil	–	–	
10	Nil	Nil	–	–	
11	Nil	Nil	–	–	
12	Nil	Nil	–	–	
13	Nil	Nil	–	–	
14	Nil	Nil	–	–	
15	10.0	14.3	24.3	12.1	
16	76.0	70.0	146.0	73.0	Typhoon "Yayang"
17	Nil	Nil	–	–	
18	Nil	2.5	2.5	1.2	
19	Nil	Nil	–	–	
20	Nil	Nil	–	–	
21	17.5	10.7	28.2	14.1	
22	69.3	65.7	135.0	67.5	
23	58.0	78.8	136.8	68.4	
24	32.2	32.0	64.2	32.1	
25	1.4	3.0	4.1	2.0	
26	3.0	1.0	4.0	2.05	
27	8.0	7.0	15.0	7.5	
28	0.1	Nil	–	–	
29	Nil	Nil	–	–	
30	Nil	Nil	–	–	
31	Nil	Nil	–	–	

Monthly precipitation in 2 stations	→ 1608.0
Total monthly average precipitation/station	→ 804.0
Average ~~daily precipitation~~	→ 26.7
RECORDED BY: *J.R. Rollo*	*VERIFIED BY:* *R.S. Alincastre*

LOCATION OF RAINGAUGE STATIONS:

1 – EDC Campsite Cabalonan
2 – TGE 11

RSA/jar:ilb

Table 1

Monthly Rainfall (in mm) for VISCA Station North of Baybay Leyte

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec
1976	421.1	199.1	269.4	40.6	115.6	261.9	161.6	419.3	128.5	125.3	146.9	468.8
1977	176.9	434.9	125.7	46.7	121.8	203.9	337.5	377.2	80.2	76.3	224.2	62.3
1978	241.7	139.3	178.8	130.3	179.9	142.0	214.7	424.2	462.9	478.4	181.5	472.7
1979	150.1	57.9	42.7	190.9	138.8	304.1	237.7	81.6	209.4	244.9	269.6	263.1
1980	356.9	277.6	57.7	149.4	24.9	357.9	271.2	511.9	298.4	191.7	506.9	230.9
1981	431.8	87.6	41.9	70.1	91.8	363.0	293.8	123.7	282.7	273.8	297.0	266.3
1982	84.3	195.3	316.6	123.5	77.2	130.1	385.9	361.6	126.9	224.1	176.6	73.0
1983	137.0	29.3	17.2	19.4	6.2	124.3	413.3	251.6	224.2	144.0	156.1	603.7
1984	334.1	415.9	179.3	137.0	60.5	83.4	165.1	194.5	122.6	252.7	221.6	371.7
1985	359.3	150.5	106.1	103.0	160.8	121.3	292.4	194.8	399.2	196.0	152.0	133.4
1986	500.1	107.8	163.5	156.6	87.2	176.9	117.7	441.6	128.0	301.2	248.4	183.1
1987	261.8	156.9	90.9	11.7	10.8	16.2	284.0	289.4	77.7	113.4	378.9	83.1
1988	134.1	134.1	64.8	183.7	56.4	251.9	113.3	184.3	201.1	252.0	329.7	253.0
1989	593.9	265.0	208.5	164.9	165.1	259.9	343.4	135.1	109.6	254.2	180.0	129.6
1990	438.0	60.7	22.7	87.2	213.8	365.5	268.5	149.8	290.1	588.0	535.0	175.4
1991	219.1	307.6	145.7	132.6	169.6	204.5	326.9	209.6	154.2	208.0	489.1	No Data
1992	100											

YELLOW CREEK FLOOD CONTROL PROJECT

by Ronald R. Copeland, Research Hydraulic Engineer, US Army Engineer Waterways Experiment Station, Vicksburg, Mississippi, and John D. (David) Hendrix, US Army Engineer District, Nashville, Tennessee

ABSTRACT

Yellow Creek is located in the southeastern Kentucky coal mining region. Its major tributaries, Stony Fork and Bennetts Fork, are steep mountain streams that rise in the Cumberland Mountains near the Kentucky-Tennessee-Virginia tri-state corner and converge onto a bowl shaped valley which contains the city of Middlesboro, Kentucky. In 1939, the U.S. Army Corps of Engineers completed construction of a 3.9-mile channel to divert the headwaters of Yellow Creek around the city. Coal strip mines and spoil areas in the rugged hills have been the source of heavy sediment loads that have historically deposited in the flat bottomed, trapezoidal bypass channel. Sediment aggradation has been so great that maintenance dredging has been conducted at least four times in the past, the most recent being in 1978-79.

A one-dimensional numerical model (TABS-1) was used to evaluate dredging options in the existing diversion channel upstream from Middlesboro, and to evaluate potential aggradation and/or degradation in a proposed channel improvement project downstream from Middlesboro. The numerical model was adjusted to simulate measured aggradation in the upstream reaches of the project; and to simulate stable conditions in the existing downstream reaches of the project. Alternative dredging cross-section options were determined using a new numerical model for hydraulic design called SAM. The design channel was then incorporated into the TABS-1 numerical model for evaluation. Maintenance requirements for the diversion channel were determined and zones of aggradation and degradation in the improved channel reaches were identified. Proposed modifications to the improved channel design were tested in the numerical model. A more efficient design in terms of sediment transport was recommended.

INTRODUCTION

Purpose and Scope

Extensive coal strip mining operations have been conducted in the uplands of the Yellow Creek watershed since the early 1950's. Much of the mine and spoil area has been abandoned and not reclaimed. Materials from these areas provide the source for the sediments deposited in the Yellow Creek Bypass channel, a U.S. Army Corps of Engineers local flood protection project. Removal of the accumulated sediment continues to be a maintenance problem. Numerical modeling techniques were employed in an effort to reduce annual maintenance costs and avoid similar problems in a newly proposed project. Specific goals of the modeling effort were to: (1) develop a dredging strategy for the existing Yellow Creek Diversion Channel; and (2) evaluate the aggradation and degradation potential of a proposed channel enlargement project for Yellow Creek immediately downstream of Middlesboro, Kentucky.

Basin Description

Yellow Creek drains approximately 60 square miles upstream of the USGS gage near the southeastern Kentucky city of Middlesboro (Figure 1). The watershed consists of steep mountains, rolling hillsides, and the nearly flat valley floor of approximately 10 mi^2. Slopes in the watershed vary from over 50 percent in the mountainous area to less than 0.2 percent in the valley. Land use in the basin is characterized by strip mining and forested areas in the uplands and almost complete urbanization in the lowlands.

The major tributaries to Yellow Creek are Stony Fork (16 mi^2) and Bennetts Fork (13 mi^2). These streams have similar basin and channel characteristics. Both rise in heavily strip mined regions of the mountains and descend at slopes of over 5 percent toward the wide, bowl-shaped valley in which the city of Middlesboro is located. Abrupt changes in the streambed slopes occur as the tributaries discharge into the bypass channel. The stream channels are nearly rectangular in shape with bed material ranging from gravels to boulders.

The Yellow Creek Bypass Channel diverts flows from Bennetts Fork, Stony Fork, and other tributaries around the city of Middlesboro. Below its confluence with the bypass channel, Yellow Creek flows through a narrow valley confined by hills that rise more than 1000 feet above the valley floor. It discharges into the Cumberland River approximately 15 miles downstream.

History of Flood Control Measures

Before Middlesboro was established, Stony Fork and Bennetts Fork combined to form Yellow Creek. As development began in 1889, the winding Yellow Creek channel was dredged and straightened through the city. Following severe flooding in 1929, the U.S. Army Corps of Engineers designed a canal and levee system to divert the waters of Stony Fork, Bennetts Fork, and other tributaries around the city. The project was authorized by the Flood Control Act of 1936 and construction of the 3.9-mile bypass channel was completed in December 1939. The flat-bottomed, trapezoidal bypass channel consists of three cuts, including one 1600-foot reach lined with concrete. Cross section bottom widths range from 80 feet to 200 feet with side slopes of 1-1.5H:1V.

It was soon realized that insufficient channel capacity downstream of the confluence of Yellow Creek and the bypass channel was causing backwater flooding in the lower parts of the city. In 1952, a clearing and snagging project was conducted to remove vegetation, gravel bars, boulders, and other debris from the nearly four-mile reach of Yellow Creek between Middlesboro and US Highway 25E. This is the same reach for which a channel enlargement project is currently proposed.

In the early 1960's, it was recognized that sediment aggredation was occurring in the upstream portion of the bypass channel. The integrity of the project was threatened after heavy deposition during floods in 1963 and 1965. A series of sediment ranges was established to monitor the condition of the project. Remedial measures were performed when subsequent surveys indicated that the bypass channel capacity was less than the project design discharge of 22,000 cfs. A summary of the dredging operations is shown in Table 1.

TABLE 1. Dredging History for Yellow Creek Bypass.

DATE	REACH (STREAM MILE)	VOLUME REMOVED (CU YDS)
1967-1968	Concrete liner to Winchester Ave (1.68-3.91)	292,000
1973-1974	1000 feet downstream of 35th St to Winchester Ave (2.59-3.91)	76,500
1977-1978	Concrete liner to 35th St (1.68-2.78)	84,000
1978-1979	35th St to Winchester Ave (2.78-3.91)	55,000
	TOTAL	507,500

Following the devastating flood of April 3-5, 1977, the Corps of Engineers was authorized to evaluate flood damage reduction measures to protect against a recurrence of that event. Although the study is currently in progress, the most favorable of the plans appears to be enlarging the Yellow Creek channel from mile 11.4 to mile 14.9.

METHODS

The sedimentation study addressed two separate issues: the proposed channel enlargement project downstream from Middlesboro, and dredging plans in the bypass channel. A combination of the SAM hydraulic design package and the TABS-1 one-dimensional sedimentation model was used to evaluate these plans. TABS-1 numerical models were developed for both the downstream and upstream reaches. The downstream model included 5.53 miles of Yellow Creek between miles 9.43 and 14.96 and then 1.37 miles of the bypass channel from its confluence with Yellow Creek to the downstream end of the concrete chute (figure 1). Initial geometry for this model was taken from 1980 Corps of Engineer surveys. The upstream model included the bypass channel upstream from the concrete chute (mile 1.7) to the mouth of Bennetts Fork (mile 3.9). Initial channel geometry for this model was taken from Corps of Engineer surveys taken after removal of sediment deposits in 1978. Surveys taken in 1987 and 1992 were used to determine sediment accumulation in the prototype. Cross-section locations and model boundaries are shown in figure 2.

Model Input Parameters

The historical histograph used in the numerical model was based on data from the US Geological Survey (USGS) gage on Yellow Creek, located at mile 11.4. Only mean daily discharges greater than 200 cfs were used, because sediment transport was found to be negligible for lessor discharges. Peak discharges, greater than 3000 cfs, were used to adjust mean daily flows to account for the increased sediment transport potential at high flow.

Bed material samples were collected from Yellow Creek and the bypass channel in March 1989. These samples indicated a wide variation in sizes (Figures 3-5). The stream channel is composed of two distinct classes of material. The low flow channel in the bypass channel is composed primarily of sand and gravel, while the bars, banks, and benches are composed primarily of fine sand and silt. Channel surveys of the bypass channel indicated that deposition occurs primarily on the bars and benches. Field observations in Yellow Creek, downstream of the bypass channel, indicated that fine sediments were depositing in slack water areas, along the banks and behind vegetation. It was also observed during the field investigation that several portions of Yellow Creek are armored with large flat cobbles. These alternate with sections of coarse sand and gravel beds in typical riffle-pools sequences. Surveys were not extensive enough to identify all of the riffles and pools. Therefore, average bed gradations were used in the numerical model. Some sections, identified as riffles during model adjustment, were assigned immobile beds to prevent excessive scour in the model.

High-water marks from a flood of 11,300 cfs were used to estimate Manning's roughness coefficients for existing channel conditions. These ranged between 0.030 and 0.043.

Available sediment inflow measurements were inadequate to define sediment inflow for the entire range of historical and design discharges. Therefore sediment inflow was used as an adjustment parameter in the numerical models. An initial estimate of sediment inflow was determined by calculating average sediment transport potential at the 5 upstream most cross sections. Sediment inflow was then reduced or increased by equal percentages in the adjustment phase of the study. Calculated outflow from the upstream model was used as inflow to the downstream model.

A modified version of the Laursen (1958) equation was used as a sediment transport function for this study. The modified Laursen equation (Laursen-Copeland) incorporates data for transport of gravels in addition to the sand data used to develop the original Laursen function. There are some differences in the way hydraulic parameters are calculated in the modified equation. This function was developed to calculate sediment transport in a sand and gravel bed stream (Copeland and Thomas 1989). A combination of the Toffaleti (1966) and Meyer-Peter and Muller (1948) equations was used to test the sensitivity of the model to transport function.

Model Adjustment

The numerical model of the bypass channel was adjusted to simulate the accumulated measured aggradation between 1978 and 1987 surveys. The model was adjusted by varying sediment inflow. Three adjusted numerical models were developed. The first was a single-grain size model (very fine sand), and sediment transport was calculated using the Laursen-Copeland function. The next two adjusted models developed were multiple grain size models which simulated sediment sizes between 0.004 and 256 mm. One of these models used the Laursen-Copeland function and the other the combined the Toffaleti and Meyer-Peter and Muller functions. Comparisons of calculated and measured longitudinal cumulative aggradation are shown in Figure 6.

The advantage of the multiple grain size models was that the longitudinal distribution of sediment deposition was closer to that measured in the prototype. In addition the adjusted sediment inflow curves were closer to the measured data. However, the calculated gradation of deposited material was much coarser than the prototype material found in the benches along the low flow channel. Appropriate quantities of fine deposition in the benches could not be obtained with the multiple-grain size models using sediment inflow concentrations extrapolated from the measured data. This is attributed to the numerical model constraint of using average hydraulic parameters.

The advantage of the single-grain size model was that the deposited material was composed of fine material just like the benches in the prototype. however, the adjusted sediment inflow at low flow was considerably higher than measured values and the longitudinal distribution of the deposit was not as good as with the multiple-grain size models. In addition, calculations with the very-fine sand model were characterized by bed elevation oscillations.

The TABS-1 numerical model uses average hydraulic parameters to calculate sediment erosion, transport, and deposition. It does not account for lateral variation in hydraulic conditions and sedimentation processes. Based on bed samples and field observations it is apparent that there are two distinct sedimentation processes occurring in the bypass channel. Coarse sand and gravel is deposited at the upstream end of the channel and at the confluence of Stony Fork. This occurs primarily at high discharges when the larger sized material can be moved. Deposition of fine sand and silt also occurs in the channel in a bench adjacent to the low flow channel. The hydraulic conditions that precipitate this bench deposit are uncertain. Flow separation and eddy development at high flow could be responsible, or deposition could occur on the recession of the flood hydrograph when flow depths and transport capacity on the bench become significantly less that in the low flow channel.

For the purpose of studying dredging alternatives, accurate replication of the longitudinal distribution of the sediment deposition was deemed to be the most important factor; and the multiple-grain size Laursen-Copeland model was chosen.

The Yellow Creek bypass channel was resurveyed in January 1992. Between 1987 and 1992, a net accumulation of about 22,400 yds^3 was calculated. About 11,400 yds^3 were eroded from the channel between the concrete chute at mile 1.70, and mile 2.1; while between miles 2.1 and 3.89, about 33,800 yds^3 were deposited.

The predictive capability of the numerical model was tested using the results from the new survey. The historical hydrograph was extended from September 1987 to September 1991, a total of 13.75 years. The numerical model was tested using two different initial geometries. The 1987 surveyed cross sections were used with the 1987-1991 hydrograph. The 1978 surveyed cross sections, adjusted to include a low-flow channel, were tested with the 1978-1991 hydrograph.

The numerical model did not predict the 1987-1992 prototype degradation that occurred between miles 1.7 and 2.1, with either initial geometry. This discrepancy is attributed to differences in bed material downstream from mile 2.1. When bed samples were collected in March 1989, no samples were obtained from this reach because water depths were too great for wadding and a boat was not available. We expect that this reach contains significantly finer bed material than the average determined from upstream samples. High runoff events could be responsible for the removal of fine material in the downstream reaches before the 1992 survey. In any event, the numerical model is not considered verified for predicting the behavior of the bed between the concrete chute and mile 2.1.

The numerical model was very successful in reproducing measured aggradation upstream from mile 2.1, as shown in Figure 7. The numerical simulation using 1987 cross sections for initial conditions reproduced 99 percent of the measured deposition. The calculated deposition between 1987 and 1991 from the numerical simulation using the 1978 cross sections for initial conditions reproduced 102 percent of the measured deposition. These results are remarkably consistent for sedimentation studies, and the numerical model is considered circumstantiated for predicting deposition upstream from mile 2.1.

The existing Yellow Creek channel downstream from the bypass channel is considered relatively stable and the downstream numerical model was adjusted to obtain minimum calculated bed changes. Several model adjustments were found to have an insignificant effect on results. These included varying the initial bed material gradations in the pool and riffle sections, varying the sediment inflow concentration from Little Yellow Creek, adjusting initial bed elevations at pools, contracting cross-sections at pools, and varying roughness coefficients with depth. Other adjustments were found to be significant and were incorporated into the model. Adjustments were made to the initial bed material gradation downstream from mile 12.13 where the model initially calculated excessive degradation. In this reach the initial bed material gradation was coarsened based on calculated bed gradations at the end of several days of high flows. Cross-sections upstream from bypass channel mile 0.49 were assigned non-erodible beds because of excessive calculated scour that was not apparent in the prototype. This assignment is reasonable due to the presence of shale bed-rock very near the thalweg elevation at mile 0.49. Non-erodible beds were also assigned to cross-sections between miles 11.37 and 11.85.

The calculated thalweg profiles with the adjusted model of the existing downstream channel at the end of the 1978-1987 simulation are shown in Figure 8. The model calculated sediment accumulation in the pools upstream from the control at mile 11.85. This accumulation was relatively rapid during the first 2 years of the simulation, but the accumulation rate decreased during the next 3 years. As the simulation proceeded the model adjusted to its incoming sediment load. The model was essentially stable after about 8000 cubic yards had deposited.

Dredging Strategy in Bypass Channel

The problem of determining the most efficient cross-section shape and an appropriate maintenance dredging strategy for the Yellow Creek bypass channel was addressed using the hydraulic design package, SAM, and the TABS-1 numerical sedimentation model. SAM was used to determine an average cross-section shape that would transport the most sediment and thereby reduce dredging requirements. This required evaluating transport capacity for various channel cross-sections, and step-wise integration of the flow duration and sediment rating curves. TABS-1 was used to calculate 10-year deposition for a 1987 base condition and for two dredging alternatives.

Composite Section Design

A composite section, with a low flow channel, is designed to provide more efficient hydraulic conditions for sediment transport at low flow without sacrificing hydraulic efficiency at high flows. The SAM program was used to compare transport efficiency for composite sections with 2, 4, and 6 ft deep low flow channels with a trapezoidal cross-section. Base widths for the low flow channel were determined by averaging the base widths of existing cross-sections upstream and downstream from the confluence with Stony Fork. The average base width upstream and downstream from the confluence with Stony Fork was 25 ft and 40 ft respectively. Side slopes of 2H:1V were assigned. Of course, other geometries may be appropriate, but using existing base widths for the low flow channel is consistent with the geomorphic principle of the stream being the best model of itself.

The average energy slope for the two reaches in the diversion channel were determined using the HEC-2 backwater model prepared from 1978 geometry. Average energy slopes were determined for the diversion channel between the concrete chute and Stony Fork, and between Stony Fork and mile 3.39 (downstream from Cumberland Avenue). In general, energy slope decreased with discharge downstream from the confluence with Stony Fork and increased with discharge upstream from the confluence with Stony Fork.

Hydraulic properties in the composite cross-section were determined separately for both the low flow channel and the bench. Sediment yield based on capacity of the cross-sections for the reach upstream of Stony Fork was calculated using the Laursen-Copeland function (Table 2).

Table 2. Sediment Yield Upstream of Stony Fork.

Section	Percent of Total Yield with Trapezoidal Section
2-ft low flow channel	136
4-ft low flow channel	153
6-ft low flow channel	155

The small increase gained using the 6-ft low flow channel does not justify extra excavation cost, and the 4-ft low flow channel was recommended for this reach. A similar analysis was conducted for the reach downstream from the confluence with Stony Fork (Table 3). The 4-ft low flow channel was also recommended for this reach.

Table 3. Sediment Yield Downstream of Stony Fork.

Section	Percent of total yield with trapezoidal section
2-ft low flow channel	101
4-ft low flow channel	118
6-ft low flow channel	124

Dredging Alternatives

The TABS-1 model was used to evaluate dredging alternatives. Dredging alternatives were evaluated by calculating deposition in the bypass channel using the 1978-1987 hydrograph, and comparing results to calculated deposition for the same 10-year hydrograph using the existing (1987) channel geometry as a base test. Dredged cross-sections for the alternatives were assigned a composite geometry with a 4-ft-deep low flow channel; 25 ft wide upstream from Stony Fork, and 40 ft wide downstream from Stony Fork. Dredging occurred between miles 2.75 and 3.48. The first alternative was to dredge about 26,000 cu yds to about the same average elevation as the 1978 dredging operation. The second alternative was to dredge about 44,100 cu yds to an average elevation about one foot lower than the 1978 dredging operation.
Aggradation in the bypass channel was calculated using the multiple-grain-size Laursen-Copeland transport function. Accumulation rates and percent increase in dredging with the dredging alternatives are shown in Table 4.

Table 4. Average Accumulation in Diversion Channel Based on 1978-1987 Hydrograph (Laursen-Copeland function).

	Calculated (cu yds)	Percent of base test	Rate 1st 3 yrs (cu yds / yr)	Rate 10 yrs (cu yds / yr)
1987 Geometry	39,800	100	5320	3980
Alternative 1	51,400	129	8180	5140
Alternative 2	57,800	145	9230	5780

Sensitivity to Transport Function

The Toffaleti & Meyer-Peter Muller multi-grain size transport function was used to evaluate model sensitivity to transport function and sediment inflow. Calculated results are shown in Table 5.

Variations between the results of this test and the test using the Laursen-Copeland function are attributed to differences in sediment inflow and transport function, and how the two functions respond to changes in channel geometry. This comparison provides a confidence interval for the numerical results.

Table 5. Average Accumulation in Diversion Channel Based on 1978-1987 Hydrograph (Toffaleti & Meyer-Peter Muller function).

	Calculated (cu yds)	Percent of base test	Rate 1st 3 yrs (cu yds / yr)	Rate 10 yrs (cu yds / yr)
1987 Geometry	34,300	100	5590	3430
Alternative 1	41,500	121	6144	4150
Alternative 2	44,200	129	6845	4420

The Meyer-Peter Muller equation was used as a single grain size transport function to evaluate the effect of the dredging alternatives on bed-load. Fine gravel (5.6 mm) was used as the representative grain size in the model. Model results showed that about 97 percent of the inflowing bed load was trapped in the diversion channel regardless of the initial geometry. For all cases tested, most of the deposition was immediately downstream from the confluence with Stony Fork and between the confluence of Stony Fork and mile 3.63. This test showed that bed load accumulation is not significantly influenced by the dredging alternatives tested.

Long-Term Trends

The long-term effect of dredging alternatives was evaluated by calculating deposition in the bypass channel using the 1978-1991 hydrograph repeated 4 times, for a 55-yr simulation. Aggradation in the bypass channel was calculated using the multiple-grain-size Laursen-Copeland transport function. The progression of total accumulated deposition in the bypass channel for the 55-year simulation is shown in Figure 10. In this figure, initial dredging volumes are indicated on the ordinate. Accumulation rates and percent increase in dredging with the dredging alternatives are shown in Table 6. Accumulation rates shown in the table are cumulative for the number of years indicated.

Table 6. Average Accumulation in Diversion Channel Based on 1978-1991 Hydrograph Repeated 4 times (Laursen-Copeland function).

	Total Calculated (yd^3)	Percent of base test	Accumulation Rate (yds^3 / year) 1st 14yrs	2nd 14yrs	3rd 14yrs	4th 14yr
1987 Geometry	235,500	100	5600	4820	3610	2790
Alternative 1	219,700	93	6170	4360	3260	1890
Alternative 2	235,500	100	6820	4380	3450	2170

Using the 1978-1991 hydrograph, it takes about 4 years for alternative 1 to fill back to predredging conditions; and about 6.5 years for alternative 2. During the first three years accumulation rates are 154 and 173 percent of the existing (1987) channel for alternatives 1 and 2 respectively. However, as the channel fills with sediment, the accumulation rates decrease, and there is less and less distinction between the alternatives. After the first 28 years, differences in accumulation rates may be on the same order or magnitude as the accuracy of the numerical model, and results must be interpreted with care. It may be concluded, however, that even though the rates decrease, the decrease is relatively small, and equilibrium conditions will not be attained in the Yellow Creek Diversion channel even after 55 years.

Standard Project Flood

The performance of dredging alternatives during the Standard Project Flood between the concrete chute and the mouth of Bennetts fork were compared using the numerical model. The sediment inflow rating curve was extrapolated beyond discharges used to circumstantiate the model. This results in some degree of uncertainty with respect to predicting actual prototype performance and must be considered when interpreting results.

Aggradation in the bypass channel during the Standard Project Flood was calculated using the multiple-grain-size Laursen-Copeland transport function. Total sediment accumulation in the bypass channel at the peak of the Standard Project Flood is shown in Figure 11. Total sediment accumulation volumes for the dredging alternatives are shown in Table 7.

Table 7. Average Accumulation in Bypass Channel Standard Project Flood (Laursen-Copeland function).

	At Peak		At End	
	Total Calculated (yd^3)	Percent of base test	Total Calculated (yd^3)	Percent of base test
1987 Geometry	71,900	100	112,700	100
Alternative 1	78,400	109	134,700	120
Alternative 2	82,800	115	140,900	125

Assessment of Downstream Design

The proposed Yellow Creek design channel, downstream from the bypass channel, calls for increasing the channel width to 100 ft through most of the study reach between miles 11.85 and 14.96; and lowering the bed elevation at mile 11.85 by about 3 ft. The numerical model's geometry was revised to account for these changes and tested with the 1978-1987 hydrograph. Increases in calculated degradation or aggradation from the existing channel were attributed to the design of the proposed channel. During the first year, 34,100 cu yds of aggradation and 3900 cu yds of degradation were calculated. After ten years, a total of 44,700 cu yds of aggradation and 11,300 cu yards of degradation were calculated. Calculated accumulated deposition for the design channel and the sediment deposition attributed to the channel improvements are shown in Figure 9. Comparing the difference in calculated accumulations between the existing and design channels, on the average, numerical model results indicate that about 32,000 cu yds of sediment deposits can be expected in the proposed channel unless an annual maintenance program is implemented. Most of this accumulation would occur during the first year.

Significant sediment accumulation was calculated in the first 0.5 mile of the improved channel. This accumulation is attributed to the decrease in sediment transport potential with the widened channel; so that it is unable to transport all of the incoming sediment load.

The numerical model of the design channel indicated that degradation would occur in reaches where the channel was not widened. At mile 13.02 the channel was not widened due to constraints by highways on both sides of Yellow Creek and about 1.1 ft of degradation was calculated. Lowering of the bed at mile 11.85, could also be a contributing factor to the degradation at mile 13.02. Degradation of about 1.6 ft was calculated at the railroad bridge at mile 14.15 where the channel retained its existing width in the proposed design. Degradation of about 3.7 ft occurred in the diversion canal upstream from its confluence with Yellow Creek. This degradation is attributed to lowering of the water-surface elevations in the improved creek channel.

Sensitivity Tests - Downstream Channel

The sensitivity of numerical model results to sediment inflow was evaluated. Envelopes of the calculated outflow from the upstream numerical model using the 1978-1987 hydrograph were used to determine high and low sediment inflow rating curves. When sediment inflow was reduced by 50 percent, sediment accumulation in the design channel after the 10 year simulation was reduced about 34 percent. When the sediment inflow was increased by 80 percent, sediment accumulation in the design channel at the end of the 10 year simulation increased by 43 percent. These results demonstrate that sediment deposition in the design channel is sensitive to sediment inflow. The general distribution of sediment is not sensitive to sediment inflow.

The numerical model was used to test the effect of decreasing the design roughness coefficients in the improved channel. In the original design, the roughness coefficients for the design channel were assigned the same value as for the natural channel. If the bank roughness is significantly higher than the bed roughness, then widening the channel should reduce the composite channel roughness coefficient. Using the Limerinos (1970) equation, and a D_{84} of 10 mm, a bed roughness of 0.022 was determined. A bank roughness of 0.050 was then calculated assuming a composite roughness of 0.038 in the existing channel. Using these values, a composite roughness coefficient of 0.030 was calculated for the improved channel. This value was incorporated into the numerical model to test the effect of the possible lower design roughness coefficients. At the end of the 10-yr simulation accumulated aggradation attributed to the design channel, with the original initial invert elevations, was about 10 percent or 3200 cu yds less with the lower roughness coefficients. An additional 1.1 ft of scour occurred in the bypass channel upstream from the confluence with Yellow Creek.

CONCLUSIONS

A 4-ft-deep low-flow channel, with a base width of 25 ft upstream from Stony Fork and 40 ft downstream was recommended. This section was found to be the most sediment transport efficient cross section, based on integration of the sediment transport and flow duration curves.

Deposition in the bypass channel for a 55-year flood and a Standard Project Flood hydrograph was calculated for a "no action" case and for two dredging alternatives. Due to the model's sensitivity to sediment inflow and channel geometry, calculated results are considered qualitative, for use in comparing alternatives, and not for predicting exact quantities. The results indicate the greater quantity of dredging, the greater the deposition rate, with deposition rates generally declining with time; but never achieving an equilibrium condition where inflowing sediment can be transported through the reach. The total amount of deposition calculated for the 55-yr period would be unexceptable due to loss of flood conveyance capacity. For purposes of calculating design water-surface elevations, it is recommended that design invert elevations be increased to account for sediment deposition.

Significant quantities of sediment will deposit in the first 0.5 mile of the proposed Yellow Creek design channel downstream from the confluence with the bypass channel, and in the existing pools at mile 12.47 and 13.35. About 32,000 cu yds can be expected on an annual basis if the channel is cleaned out every year. Sediment accumulation should be accounted for in design water-surface calculations, in the absence of an annual cleanout program.

Scour will occur in the bypass channel upstream from the confluence with Yellow Creek. This can be corrected with a head cut control structure. Otherwise, degradation may be accompanied by lateral migration of the stream which could threaten existing levees and adjacent property.

Scour will also occur at the railroad bridge and at mile 13.02 where the design channel is constricted. Structures at these locations should be considered for local scour protection.

The nature of the channel bed in the vicinity of mile 11.85 should be thoroughly investigated. Evidence from this investigation indicates that the existing bed is composed of resistant material that controls stream response upstream. If the proposed cut exposes a less-resistant layer of material, severe channel unraveling could occur. This can be corrected, if necessary, with a control structure.

REFERENCES

Copeland, R. R., and Thomas, W. A. 1989. "Corte Madera Creek Sedimentation Study - Numerical Model Investigation," Technical Report HL-89-6, US Army Engineer Waterways Experiment Station, Vicksburg, Miss.

Laursen, Emmett M. 1958 (February). "The Total Sediment Load of Streams," Journal, Hydraulics Division, American Society of Civil Engineers, Vol 84, No. HY1, pp1530-1 to 1530-36

Limerinos, J. T. 1970. "Determination of the Manning Coefficient from Measured Bed Roughness in Natural Channels," Geological Survey Water-Supply Paper 1989-B, US Geological Survey, Washington, DC.

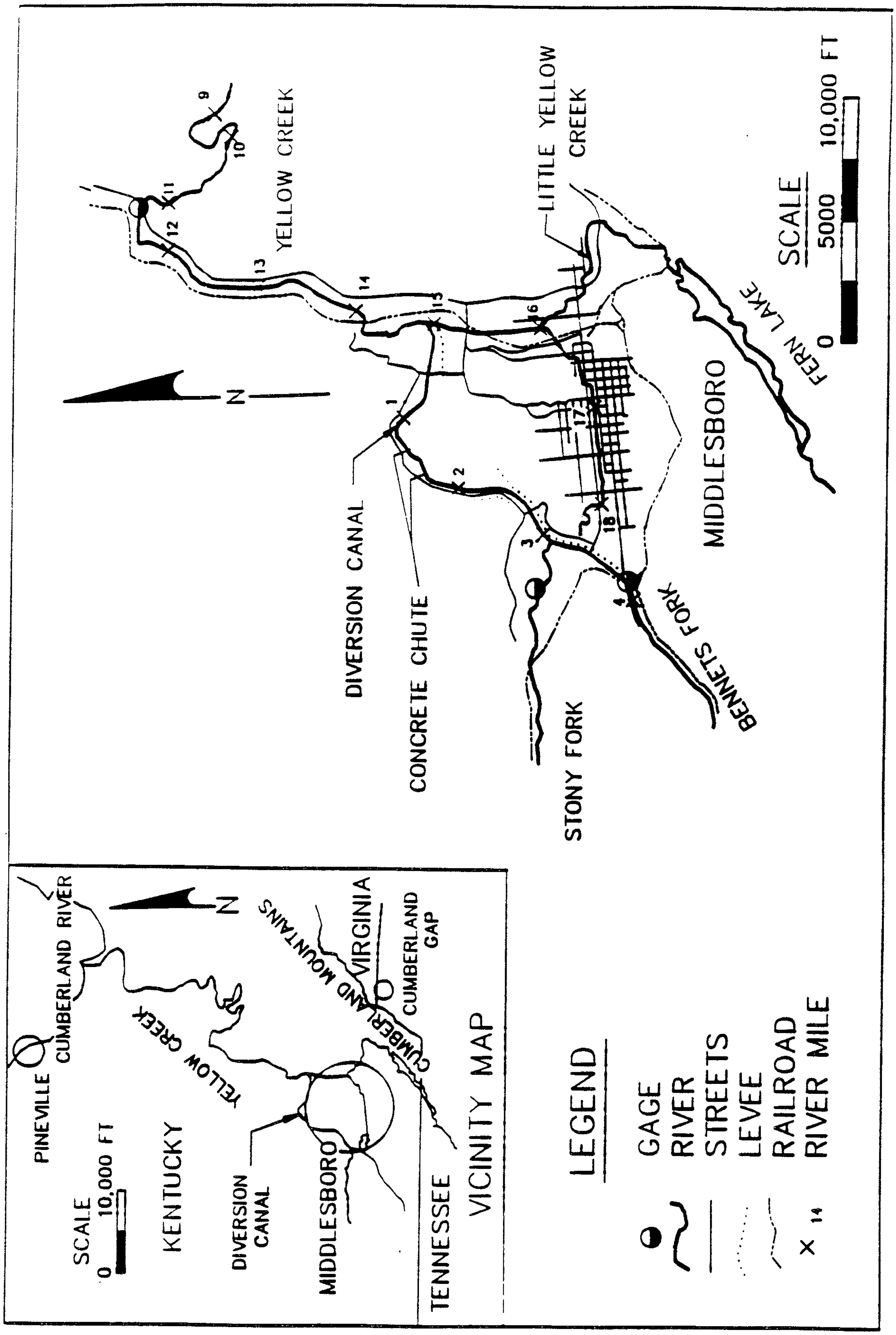
Meyer-Peter, E., and Muller, R. 1948. "Formulas for Bed Load Transport," Report on Second Meeting of International Association for Hydraulic Research, Stockholm, Sweden, pp 39-64.

Toffaleti, F. B. 1966 (November). "A Procedure for Computation of Total River Sand Discharge and Detailed Distribution, Bed to Surface," Committee on Channel Stabilization, US Army Corps of Engineers, Washington, DC.

USAED Nashville 1966. "Cumberland River Basin, Middlesboro, Kentucky, Local Protection Project - Design Memorandum No. 1," August.

__________ 1971. "Flood Plain Information; Yellow Creek and Little Yellow Creek, Bennetts and Stony Forks, and Diversion Canal; Middlesboro Kentucky," Prepared for the City of Middlesboro, Nashville, Tennessee.

US Army Engineer Hydrologic Engineer Center. 1991. "Users Manual HEC6 Generalized Computer Program Scour and Deposition in Rivers and Reservoirs," US Army Corps of Engineers, Davis Calif.



YELLOW CREEK
LITTLE YELLOW CREEK
SCALE
0 5000 10,000 FT
N
FERN LAKE
MIDDLESBORO
DIVERSION CANAL
CONCRETE CHUTE
STONY FORK
BENNETS FORK
PINEVILLE
CUMBERLAND RIVER
CUMBERLAND MOUNTAINS
VIRGINIA
CUMBERLAND GAP
YELLOW CREEK
SCALE
0 10,000 FT
KENTUCKY
DIVERSION CANAL
MIDDLESBORO
TENNESSEE
VICINITY MAP
LEGEND
GAGE
RIVER
STREETS
LEVEE
RAILROAD
RIVER MILE
14

Figure 1

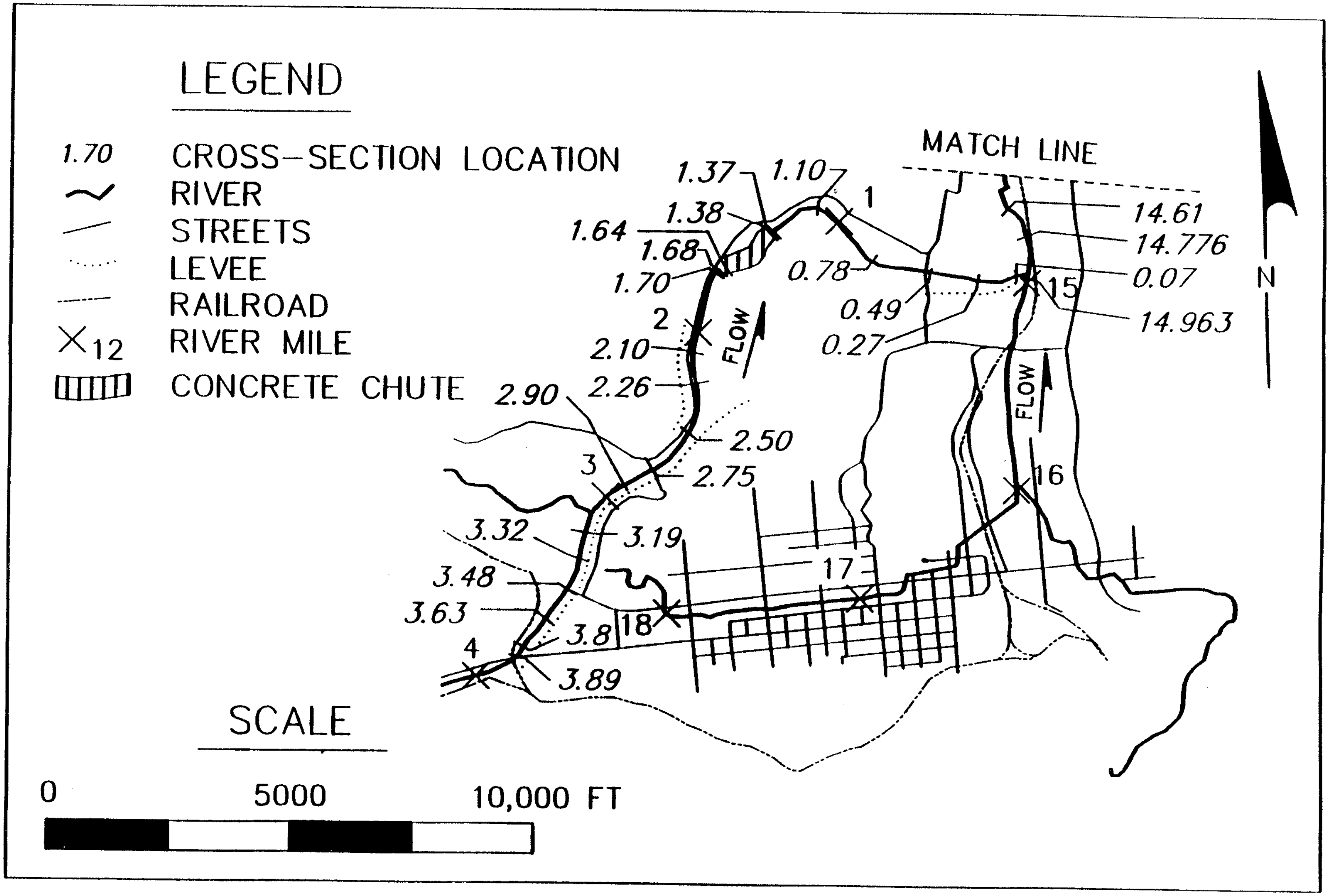
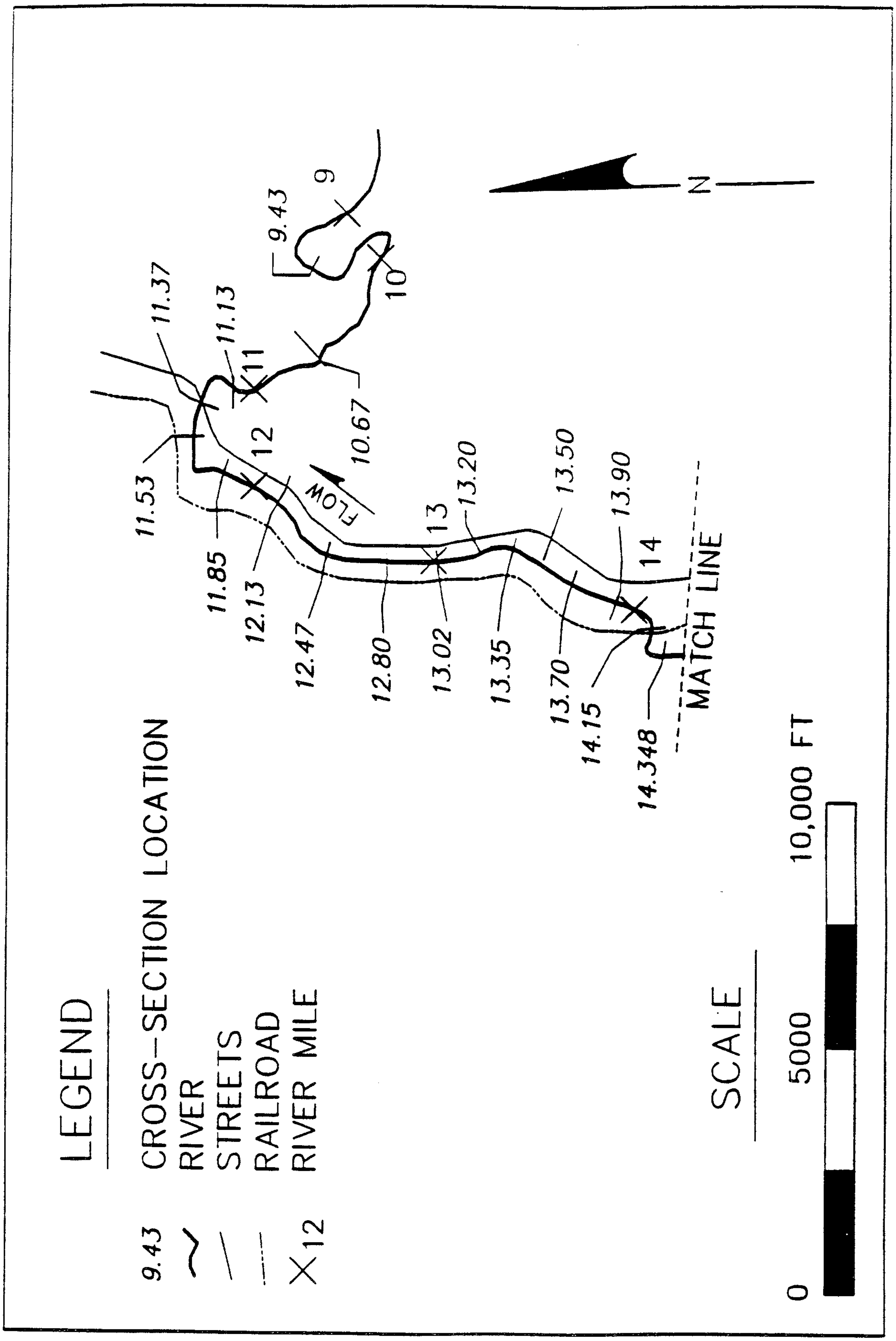
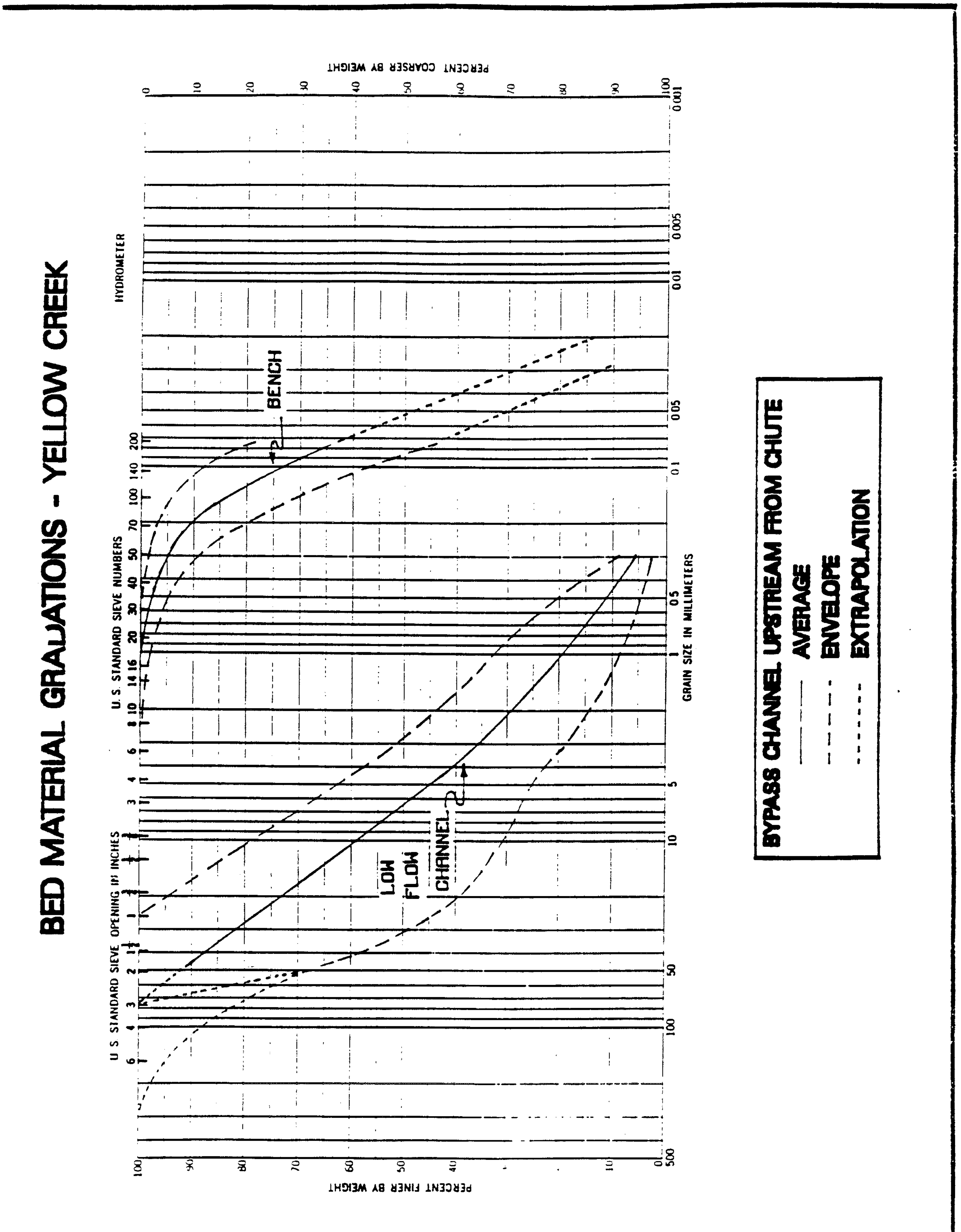

LEGEND
1.70 CROSS-SECTION LOCATION
RIVER
STREETS
LEVEE
RAILROAD
12 RIVER MILE
CONCRETE CHUTE
MATCH LINE
N
FLOW
FLOW
1.37
1.10
1
1.38
1.64
1.68
1.70
0.78
0.49
0.27
14.61
14.776
0.07
15
14.963
2
2.10
2.26
2.90
2.50
3
2.75
16
3.32
3.19
17
3.48
3.63
18
3.8
4
3.89
SCALE
0
5000
10,000 FT

Figure 2a



LEGEND
9.43 CROSS-SECTION LOCATION
RIVER
STREETS
RAILROAD
12 RIVER MILE
SCALE
0
5000
10,000 FT
N
FLOW
MATCH LINE
9
9.43
10
10.67
11
11.13
11.37
11.53
11.85
12
12.13
12.47
12.80
13
13.02
13.20
13.35
13.50
13.70
13.90
14
14.15
14.348

Figure 2b



BED MATERIAL GRADATIONS - YELLOW CREEK
U.S. STANDARD SIEVE OPENING IN INCHES
U.S. STANDARD SIEVE NUMBERS
HYDROMETER
PERCENT COARSER BY WEIGHT
PERCENT FINER BY WEIGHT
GRAIN SIZE IN MILLIMETERS
BENCH
LOW FLOW CHANNEL
BYPASS CHANNEL UPSTREAM FROM CHUTE
AVERAGE
ENVELOPE
EXTRAPOLATION

Figure 3

BED MATERIAL GRADATIONS - YELLOW CREEK

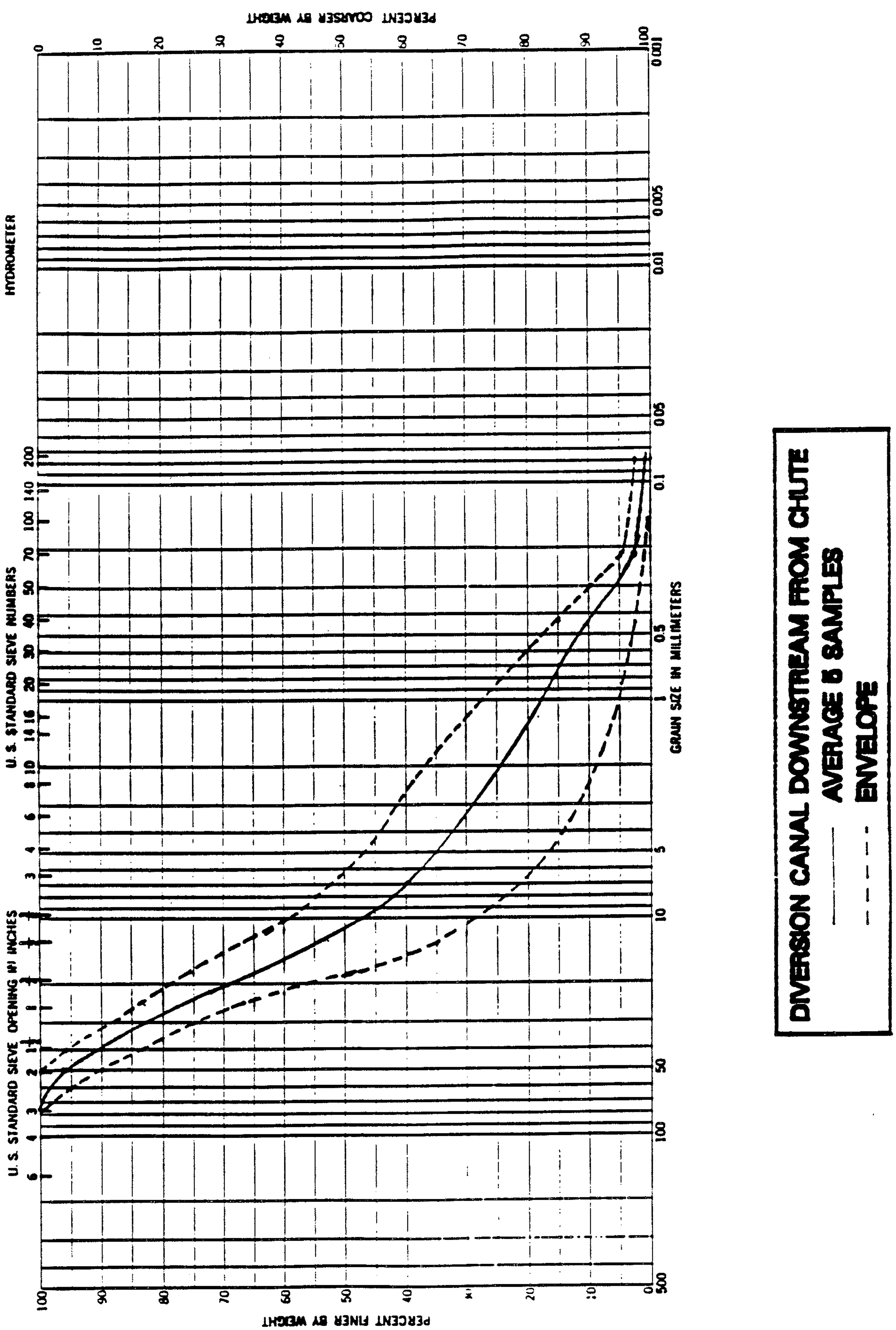
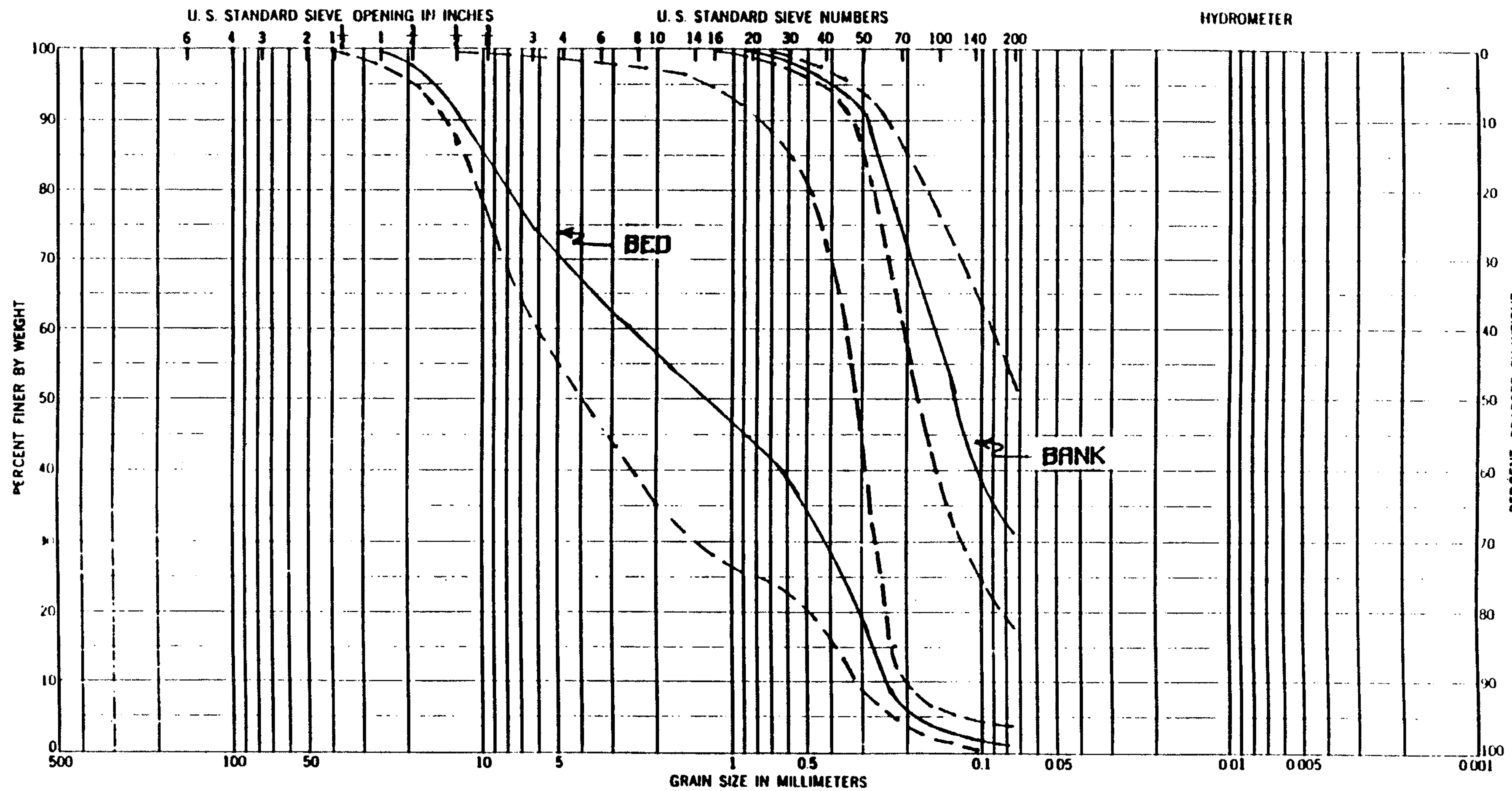


Figure 4

BED MATERIAL GRADATIONS - YELLOW CREEK



YELLOW CREEK CHANNEL AND BANKS

—·— AVERAGE

- - - - ENVELOPE

Figure 5

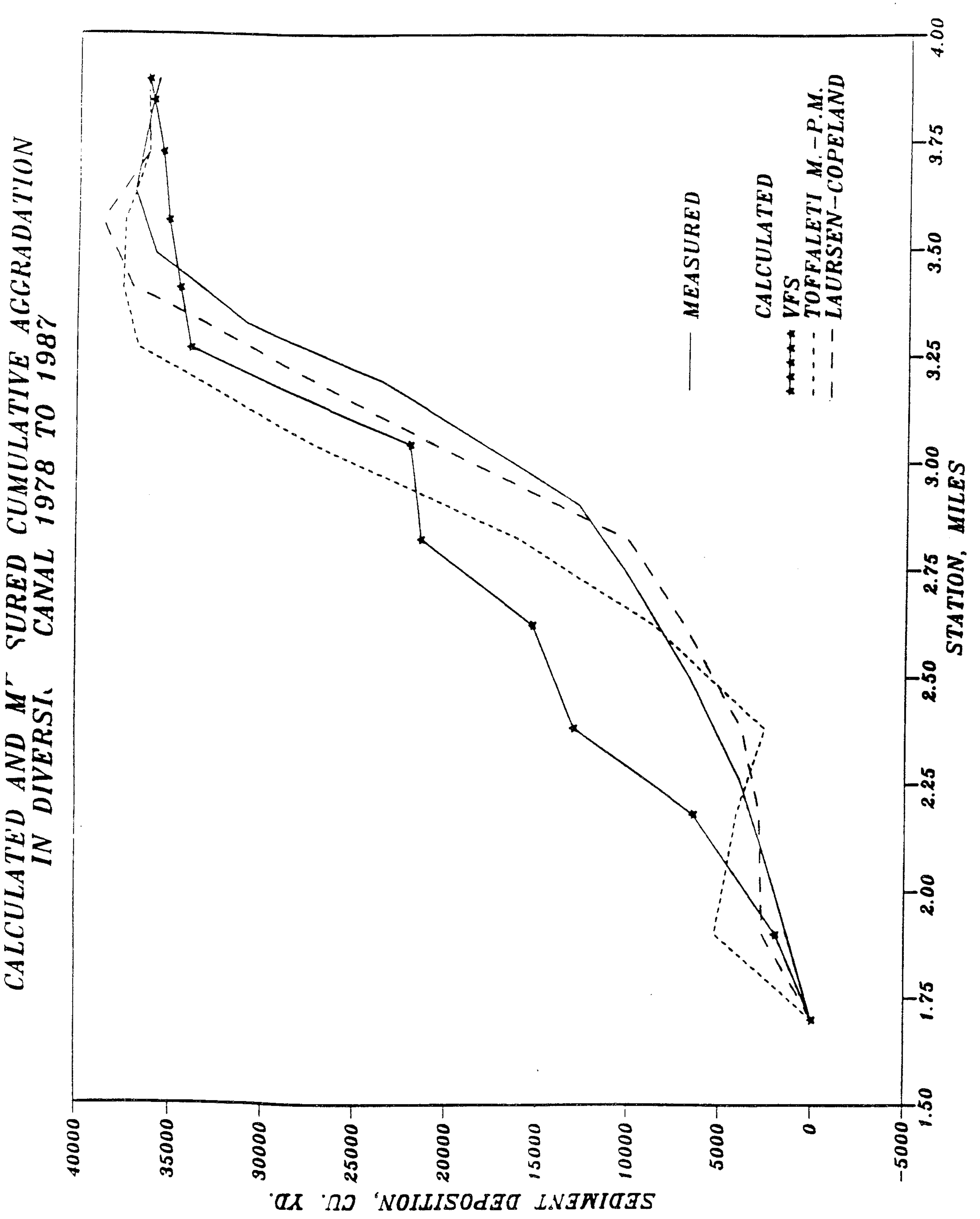

CALCULATED AND M SURED CUMULATIVE AGGRADATION
IN DIVERS CANAL 1978 TO 1987
MEASURED
CALCULATED
VFS
TOFFALETI M.-P.M.
LAURSEN-COPELAND
STATION, MILES
SEDIMENT DEPOSITION, CU. YD.
-5000
0
5000
10000
15000
20000
25000
30000
35000
40000
1.50
1.75
2.00
2.25
2.50
2.75
3.00
3.25
3.50
3.75
4.00

Figure 6

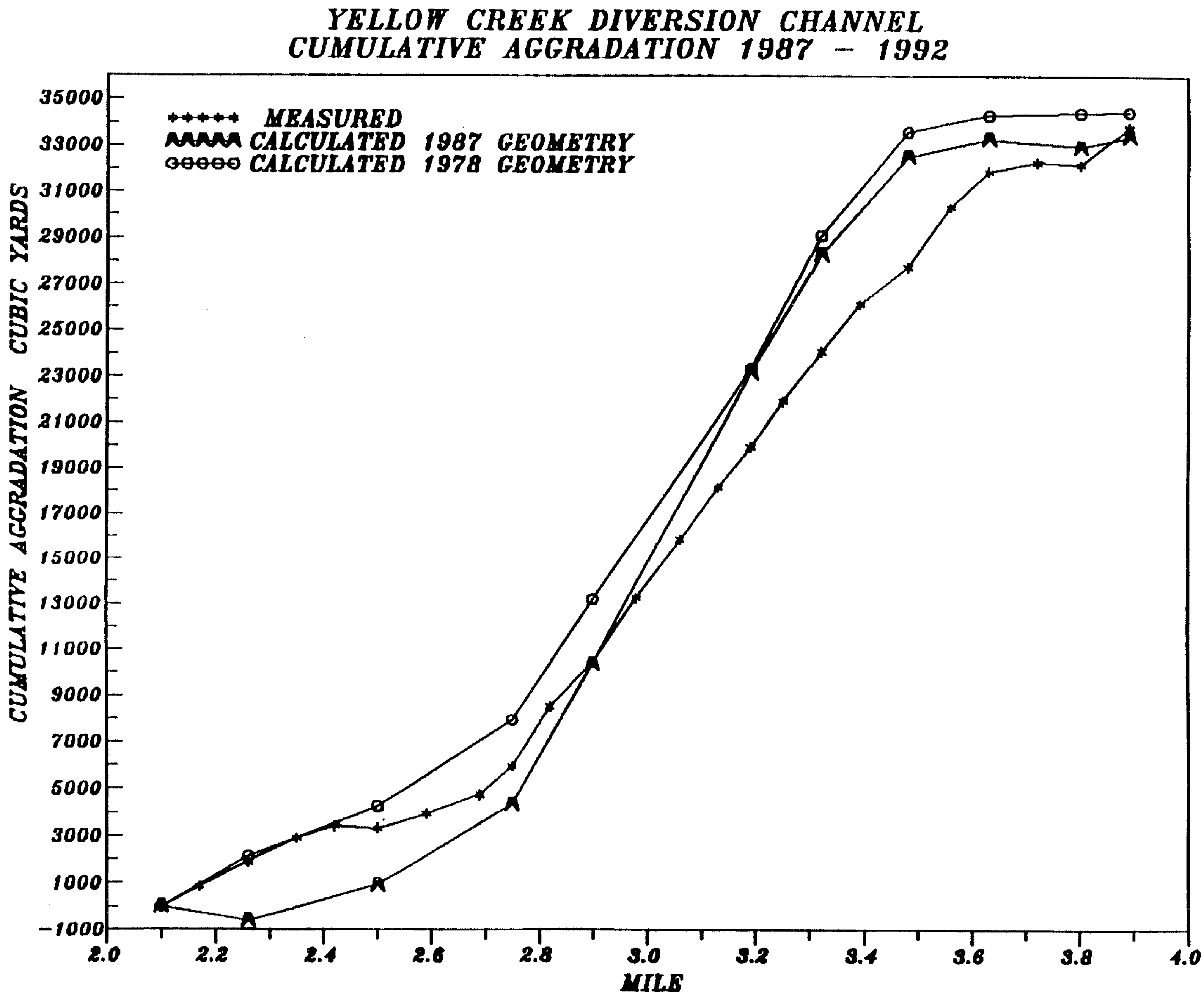

YELLOW CREEK DIVERSION CHANNEL
CUMULATIVE AGGRADATION 1987 - 1992
MEASURED
CALCULATED 1987 GEOMETRY
CALCULATED 1978 GEOMETRY
CUMULATIVE AGGRADATION CUBIC YARDS
35000
33000
31000
29000
27000
25000
23000
21000
19000
17000
15000
13000
11000
9000
7000
5000
3000
1000
-1000
2.0
2.2
2.4
2.6
2.8
3.0
3.2
3.4
3.6
3.8
4.0
MILE

Figure 7

CALCULATED BED ELEVATION CHANGE IN EXISTING CHANNEL AFTER 1978-1987 HYDROGRAPH

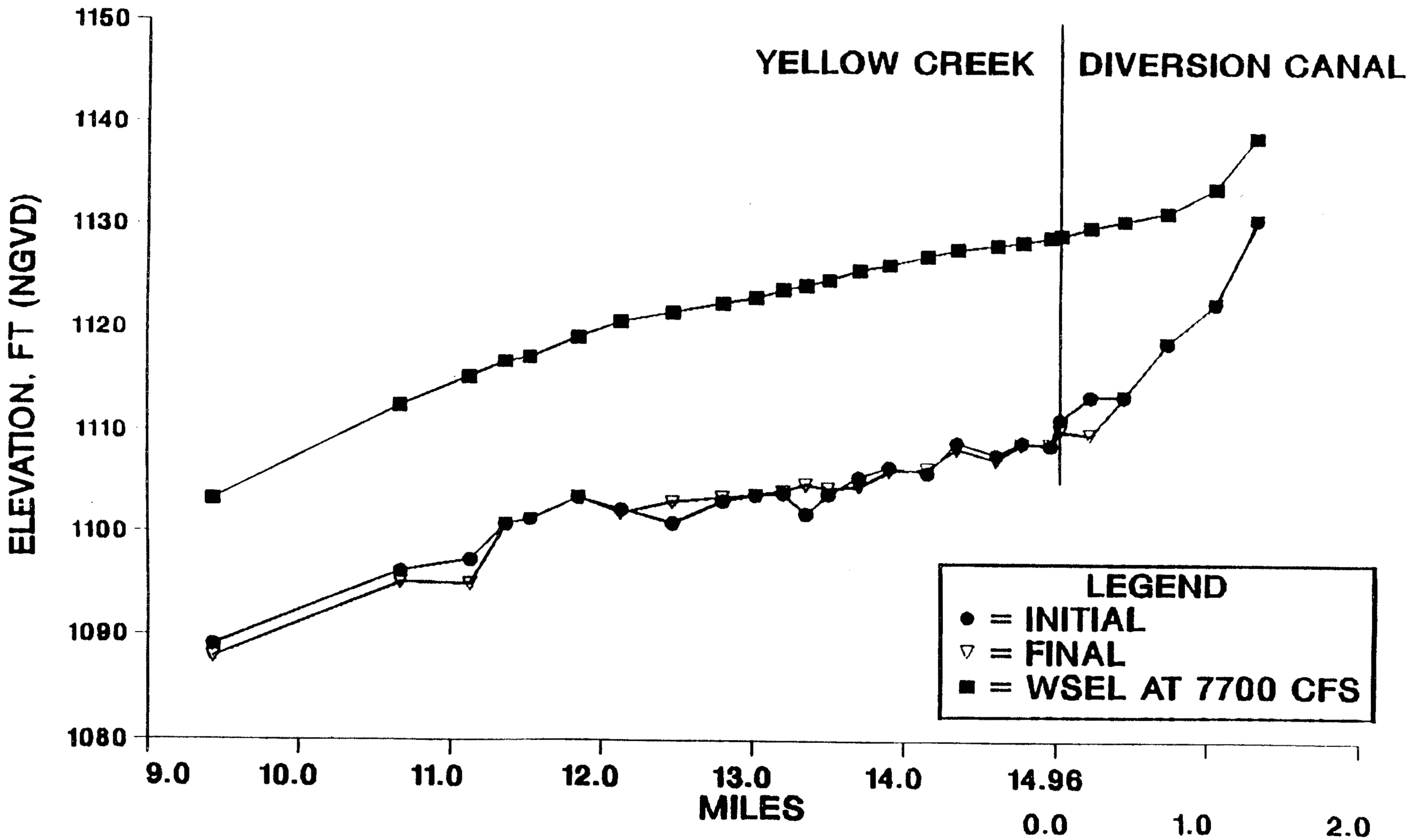


Figure 8

CALCULATED NET SEDIMENT ACCUMULATION BETWEEN MILES 11.53 AND 14.96 FOR EXISTING AND DESIGN CHANNELS

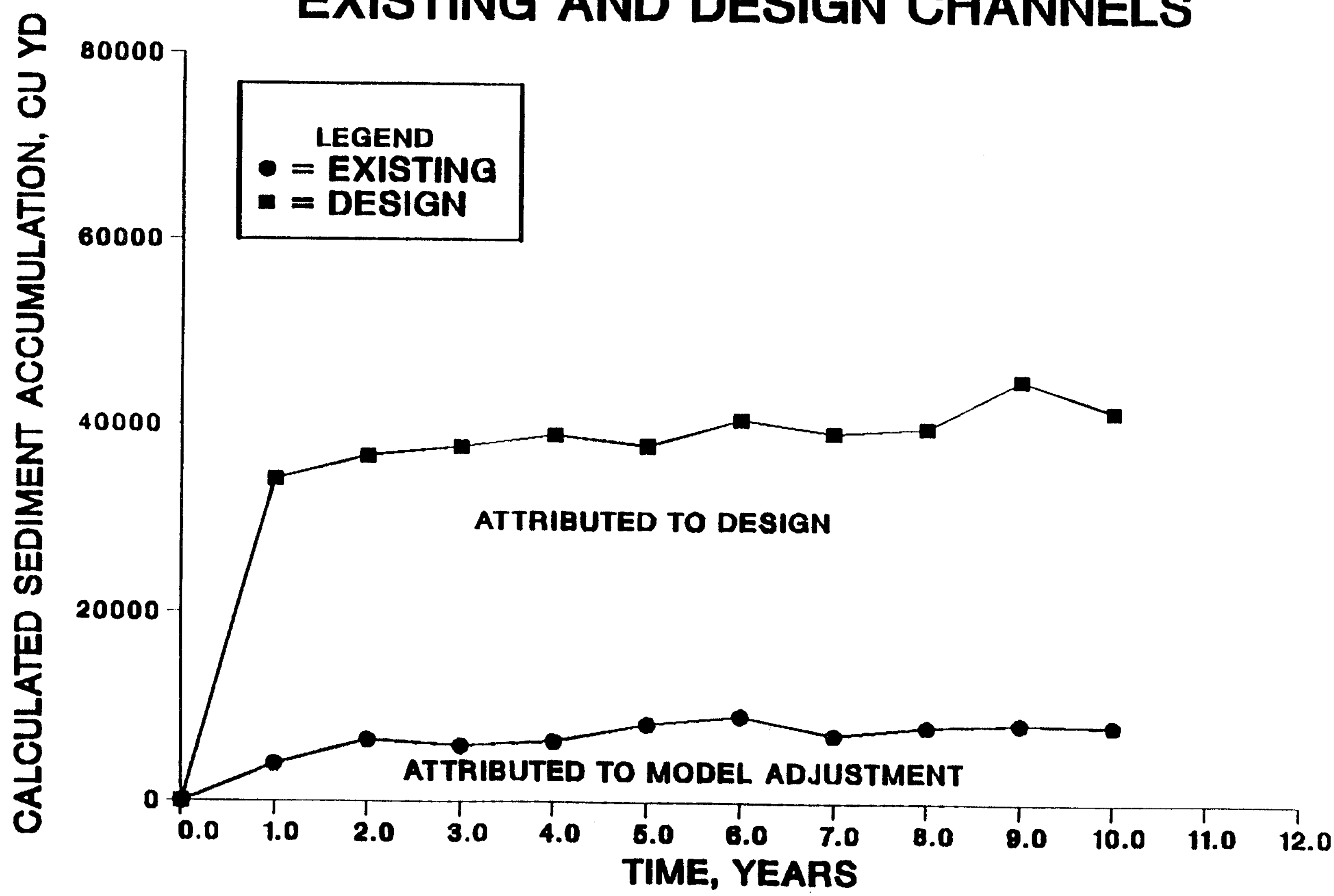
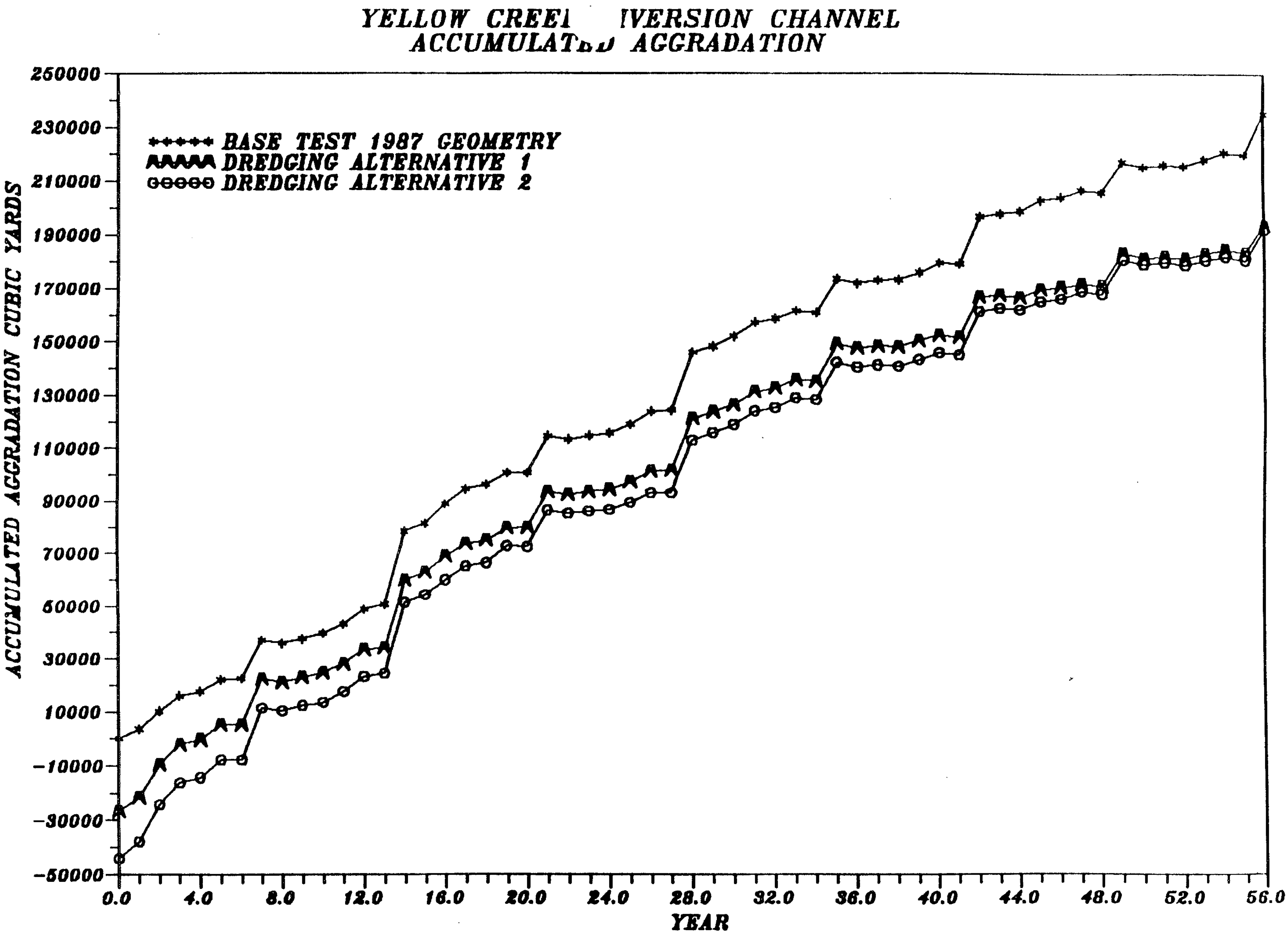


Figure 9



YELLOW CREEI IVERSION CHANNEL
ACCUMULAT AGGRADATION
BASE TEST 1987 GEOMETRY
DREDGING ALTERNATIVE 1
DREDGING ALTERNATIVE 2
ACCUMULATED AGGRADATION CUBIC YARDS
250000
230000
210000
190000
170000
150000
130000
110000
90000
70000
50000
30000
10000
-10000
-30000
-50000
0.0
4.0
8.0
12.0
16.0
20.0
24.0
28.0
32.0
36.0
40.0
44.0
48.0
52.0
56.0
YEAR

Figure 10

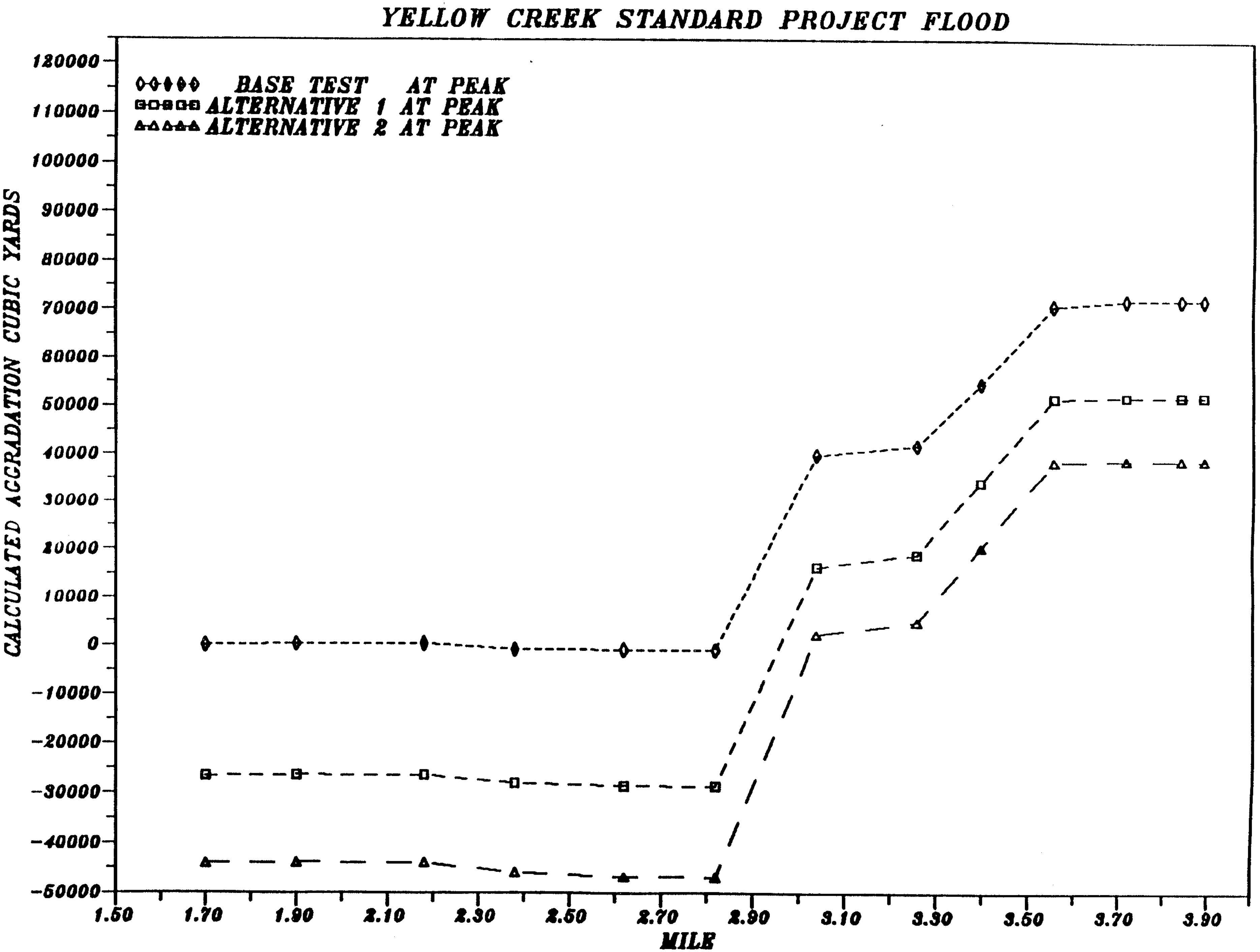

YELLOW CREEK STANDARD PROJECT FLOOD
BASE TEST AT PEAK
ALTERNATIVE 1 AT PEAK
ALTERNATIVE 2 AT PEAK
CALCULATED AGGRADATION CUBIC YARDS
120000
110000
100000
90000
80000
70000
60000
50000
40000
30000
20000
10000
0
-10000
-20000
-30000
-40000
-50000
1.50
1.70
1.90
2.10
2.30
2.50
2.70
2.90
3.10
3.30
3.50
3.70
3.90
MILE

Figure 11

IAO STREAM FLOOD CONTROL PROJECT
WAILUKU, MAUI, HAWAII
October 1992

by James Pennaz
Hydraulic Engineer
U.S. Army Engineer Division, Pacific Ocean

GENERAL

The Iao Stream Flood Control Project was authorized under Section 203 of the Flood Control Act of 1968 (PL 90-483). Project construction was initiated in April 1977 and completed in May 1981. The project is located near Wailuku, Maui and consists of a debris basin located 2.5 miles upstream from the stream mouth, channel improvements extending 3,500 feet downstream from the debris basin, levees along the right bank and floodplain management along the left bank for 6,950 feet of natural stream, and stream realignment with channel improvements for a reach of 1,730 feet near the mouth (Figure 2). Iao Stream is a steep (140 feet/mile) stream with a cobble and boulder bed. Floods during construction and shortly after construction caused scour along the right bank levees with depths exceeding six feet under the slope lining. This paper will present a case study of the project construction, subsequent scour problems, and proposed remediation.

PROJECT CONSTRUCTION

A flood in December 1980 (during project construction) caused extensive erosion to a sacrificial berm. Levee undermining also occurred from the flood along the right bank from stream mile 0.8 to 1.5. After the flood, the construction contract was modified to extend the levees into the streambed and add boulder concrete lining. Shortly after construction, additional erosion occurred to the levee toe with scour depths extending to six feet below the boulder concrete slope lining. In January 1983, additional corrective work to the levees was authorized to extend the boulder concrete slope protection to five feet below the eroded stream bottom. This work was completed in November 1983. In August 1989, the County of Maui requested additional corrective work for still more erosion problems resulting from storm events since the November 1983 corrective work. Subsequent storms had again eroded the stream bed below the levee toe by four to six feet.

SCOUR PROBLEMS

Iao Stream drains a steep valley with an abundance of sediments produced by erosional forces. Bed material size ranges from sands and gravels to boulders. The stream bed becomes lined with boulders due to "vibrational" effects of low flows moving smaller sized material into voids between the larger boulders.

A debris basin with a capacity of 80,000 cubic yards is located at the upstream end of the project. The debris basin traps nearly all sediment entering the project area for channel discharges less than 3,000 cfs. For discharges greater than 3,000 cfs, an unknown amount of sediment passes from the debris basin into the project area. A sediment delta has formed within the debris basin that is four feet high by forty feet long by seventy feet wide.

Iao stream has an energy surplus that begins to erode the unlined channel at the downstream end of the concrete channel improvement near stream mile 1.7. Velocities in this area range from fourteen to twenty eight feet per second. The natural, unprotected left bank erodes and leaves a talus deposit. The talus deposit naturally protects the left bank and redirects low flows into the right bank levee toe, thereby increasing erosion potential. The stream channel also becomes narrower on the right side as the stream bed erodes. The narrower stream has increased velocities that tend to reinforce the erosion potential to the right bank levee toe sections.

Stream discharges in the floodplain and levee area are shown on Table 1.

TABLE 1
STREAM DISCHARGES - IAO STREAM
AT RIGHT BANK LEVEES
DRAINAGE AREA = 9.4 SQ. MI.

Exceedence Frequency Per Hundred Years	Discharge (cfs)
50	2,760
20	5,700
10	8,000
5	10,600
2	15,000
1	19,600
Standard Project Flood	26,500

Discharges for flood events since construction was initiated in April 1977 have been less than the 5-year event. However, these historical floods as shown in Table 2 have caused the extensive levee toe erosion evident in annual inspections.

TABLE 2
SELECTED IAO STREAM DISCHARGES FROM 1977 TO 1992

Date	Discharge (cfs)
November 12, 1978	4,660
December 8, 1980	4,300
January 21, 1982	3,380
December 25, 1983	3,400
November 18, 1985	3,750
May 5, 1987	4,720
November 4, 1988	4,420
January 28, 1988	5,570
November 8, 1989	3,840
January 27, 1991	4,520
October 9, 1991	4,380

PROPOSED REMEDIATION

Initial recommendations from the Waterways Experiment Station include the following options: Widen the basal channel, allow additional flood water to exit the main channel area to the left bank floodplain, relocate or selectively remove channel bar/bed sediments, reduce the natural left descending bank slope, and concrete the levee toe area (Figure 3).

Widening the basal channel would reduce flow velocities and the resulting energy reduction would retard the erosion process. Allowing additional water to exit the main channel area also would reduce eroding forces in the main channel. The channel bar/bed sediment relocation would place material against the levee toe and concrete the material in place to prolong the toe area life.

Reducing the natural left descending bank slope to three horizontal to one vertical would slow the redevelopment of talus deposits and the consequent reduction in basal channel width. Vegetating the slope will increase bank stability and induce a higher hydraulic roughness factor.

ACKNOWLEDGMENT

Helen Stupplebeen, Civil Engineer, Pacific Ocean Division prepared the "Modifications to Completed Project Report, Iao Stream Flood Control Project". Dr. Monte Pearson from the Waterways Experiment Station visited the site and made the initial recommendations.

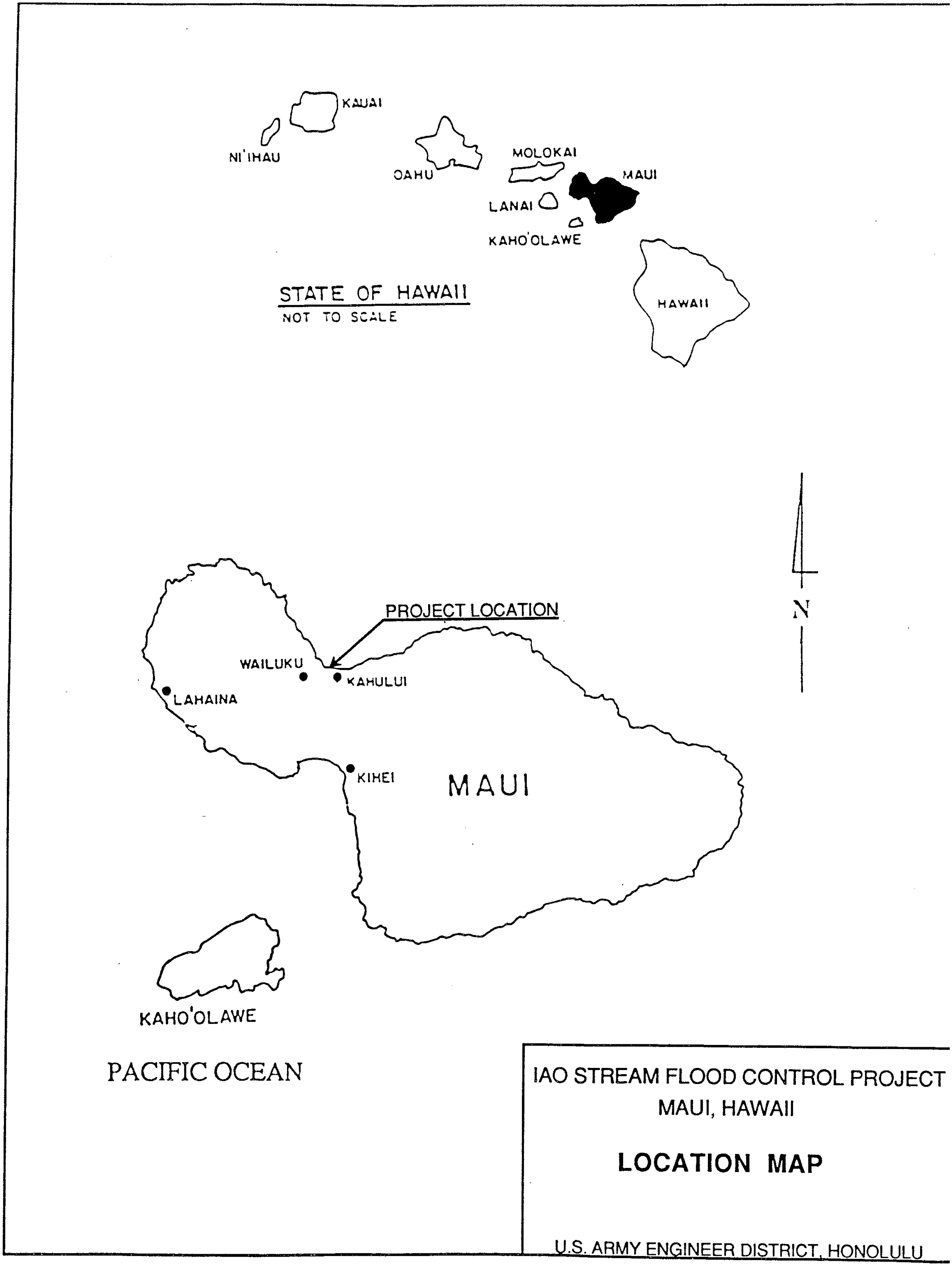

KAUAI
NI'IHAU
OAHU
MOLOKAI
MAUI
LANAI
KAHO'OLAWE
HAWAII
STATE OF HAWAII
NOT TO SCALE
PROJECT LOCATION
WAILUKU
KAHULUI
LAHAINA
KIHEI
MAUI
N
KAHO'OLAWE
PACIFIC OCEAN
IAO STREAM FLOOD CONTROL PROJECT
MAUI, HAWAII
LOCATION MAP
U.S. ARMY ENGINEER DISTRICT, HONOLULU

Figure 1

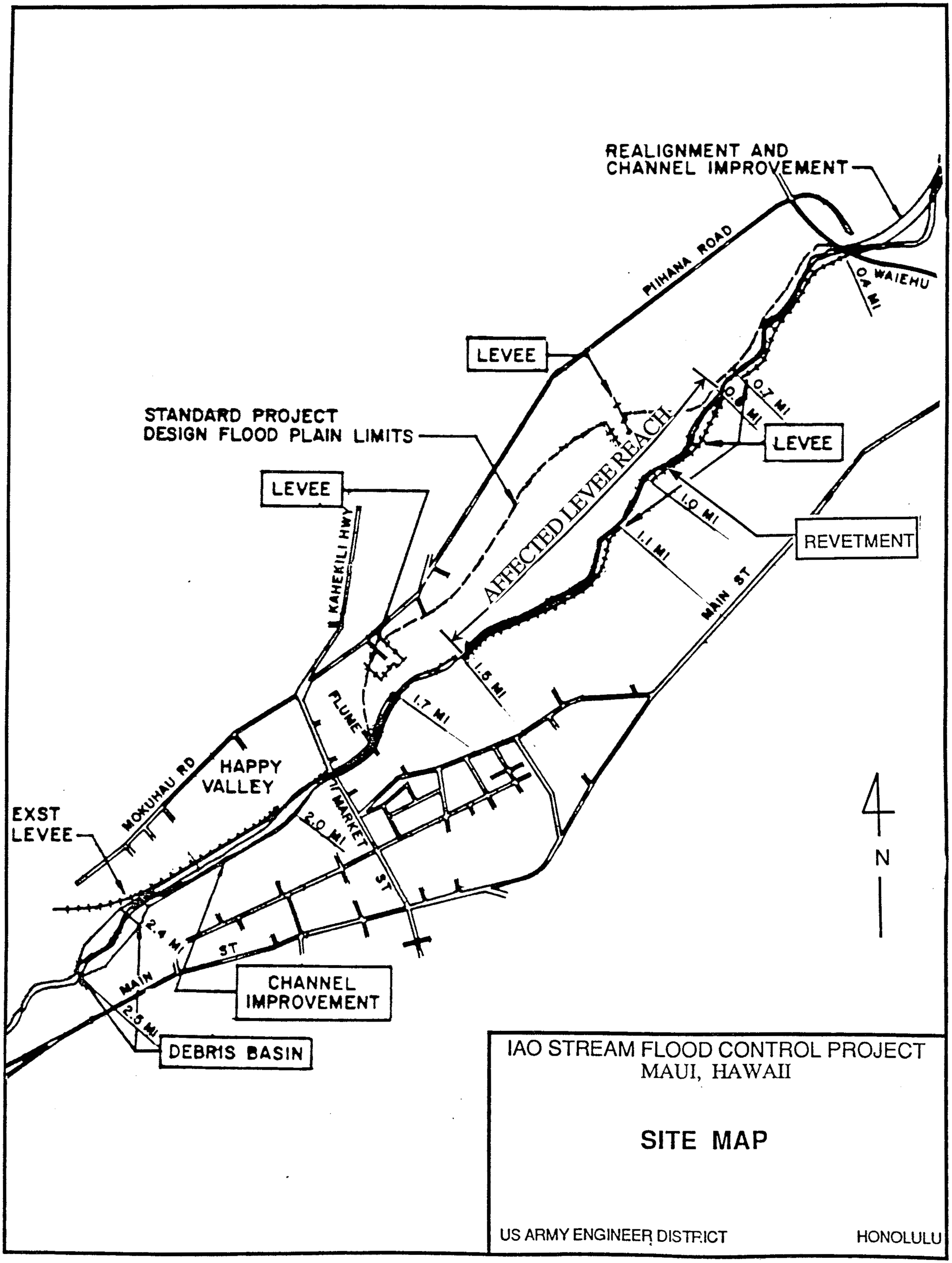
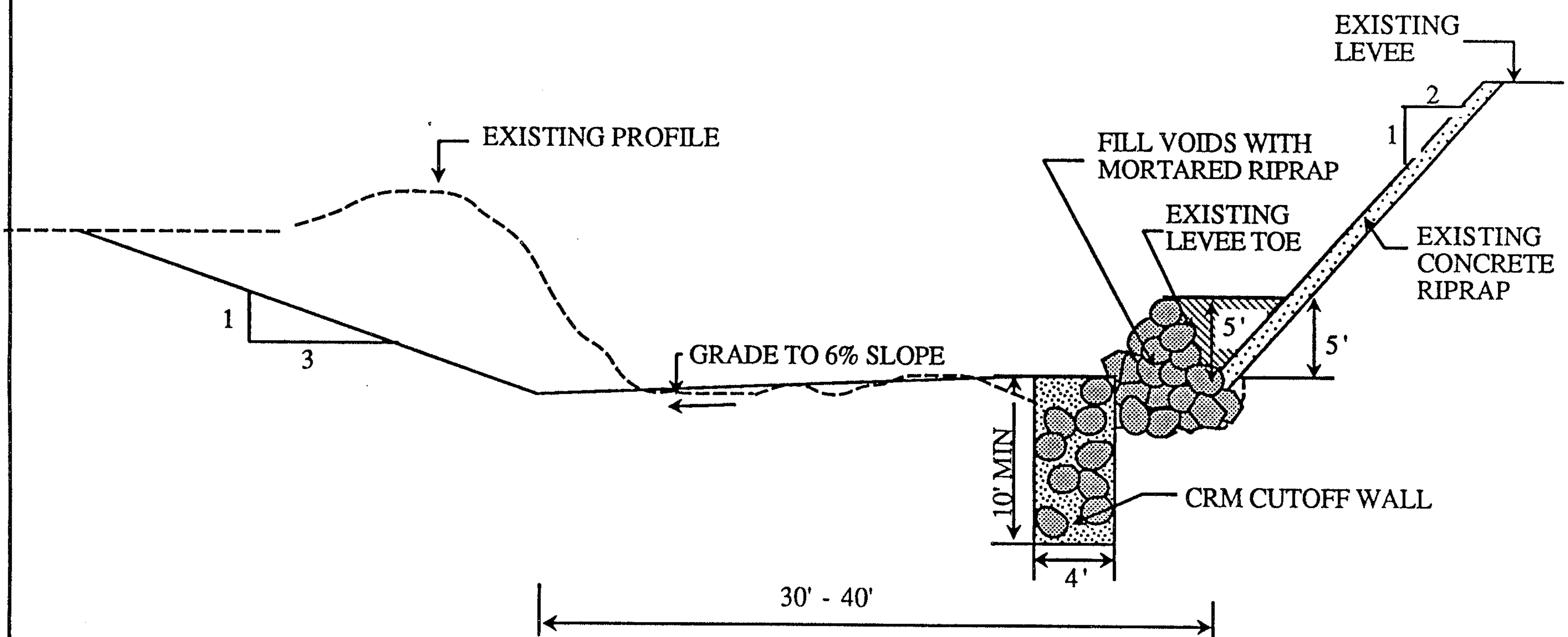

REALIGNMENT AND CHANNEL IMPROVEMENT
PIIHANA ROAD
WAIEHU
0.4 MI
LEVEE
STANDARD PROJECT DESIGN FLOOD PLAIN LIMITS
AFFECTED LEVEE REACH
0.7 MI
LEVEE
LEVEE
KAHEKILI HWY
1.0 MI
REVETMENT
1.1 MI
MAIN ST
1.5 MI
FLUME
1.7 MI
HAPPY VALLEY
MOKUHAU RD
EXST LEVEE
MARKET ST
2.0 MI
2.4 MI
ST
MAIN
2.5 MI
CHANNEL IMPROVEMENT
DEBRIS BASIN
N
IAO STREAM FLOOD CONTROL PROJECT
MAUI, HAWAII
SITE MAP
US ARMY ENGINEER DISTRICT
HONOLULU

Figure 2



SCALE 1" = 10'

IAO STREAM FLOOD CONTROL PROJECT
MAUI, HAWAII

TYPICAL MODIFIED SECTION

US ARMY ENGINEER DISTRICT HONOLULU

Figure 3

Abstract

Special Hazards Associated With Steep Channel Catchments: Debris Flows, Debris Torrents and Lahars

by Robert C. MacArthur, Principal
Resource Consultants and Engineers, Inc.
1477 Drew Avenue, Suite No. 107
Davis, CA 95616

All rivers are subject to flooding risks. Flat gradient, fine grained rivers have different geomorphic and flow characteristics than do steep, coarse grained channels and, therefore, behave differently during significant flooding events. The energy per unit flow is much greater in steep channels and, therefore, the ability to carry large volumes of sediment and debris is much greater. Steep channel floods are usually more episodic, produce greater velocities and impact forces and have the ability to carry large diameter materials and higher concentrations of sediment and debris than flat gradient rivers. The net effect and overall magnitude of flooding in steep catchments is not only a function of storm intensity but is also directly related to antecedent watershed conditions and geomorphology. It is therefore, very difficult to assign traditional hydrologic frequencies to this class of special hazards.

This paper discusses the hydrologic, morphologic and sediment characteristics necessary for triggering special flooding hazards such as debris flows, debris torrents and lahars. It discusses where such special hazards typically occur and why they are difficult to predict, manage and design for. Unique hydraulic and rheologic characteristics of debris flows and lahars are presented along with discussions of where and why most "traditional assessment procedures" fail to quantify the true nature and magnitude of this special class of flooding hazard. Finally, the paper suggests a "Hybrid Assessment and Modeling Procedure" for evaluating debris flows, debris torrents and lahars. References to the international literature and to a wide range of case histories of past floods involving hazards are provided.

Abstract

Field Estimation of Resistance Coefficients for Gravel-Bedded Stream Channels

by Charles L. Rosenfield, Ph.D.
Department of Geosciences
Oregon State University

And

Monte L. Pearson, Ph.D.
USAE Waterways Experiment Station
Vicksburg, MS 39181

Many methods are available for field estimation of resistance coefficients. most are expressed in terms of Manning's 'n,' although Chezy's flow resistance factor 'C' is also frequently used. This investigation proposes a 'component' resistance estimation technique which examines the characteristics of the channel, and includes the effects of streamside vegetation, major obstructions or flow deflectors, channel form and bedform characteristics. The investigation was performed at the Oak Creek Experimental Watershed at Oregon State University. The study was sponsored by the Geotechnical Laboratory of the Corps of Engineers, Waterways Experiment Station.

Abstract
Flood Control on Two Gravel Bed Streams at Seward, Alaska

by David Mierzejewski
Alaska District
US Army Corps of Engineers

Lowell and Fourth of July Creeks are steep gravel bed streams that flow from the rugged Kenai Mountain Range on the north gulf coast of Alaska. Both streams have well developed alluvial fans with major industrial facilities and the town of Seward built on them. Both are protected by flood control projects. The Lowell Creek flood control project was built by the Corps of Engineers in 1940 and uses a diversion dam and tunnel to divert flow off its fan. The Fourth of July Creek project was built by the City of Seward in 1980 and consists of a levee to keep flows off the developed portion of the fan. A major flood event occurred in the Seward area in 1986. The maximum 24 hour precipitation observed during the event related to a 350 year return interval. Both projects performed well but were severely damaged. The streams are subject to damaging by debris flow and landslides with subsequent failure of the dams and release of a flood wave. The 1986 storm caused this type of event to occur on several streams in the Seward area. Fourth of July Creek had an estimated discharge of 30,000 cubic feet per second from a 14 square mile subbasin due to a landslide dam breach. Tree ring dating was used to estimate the return interval for these type of events. Results indicated major sediment movement on fans in the area may occur every 75-150 years. The tree ring data implies that this type of flood event has a major impact on the magnitude of floods in the area. Annual peak frequency curves for both streams were adjusted to reflect this distinct population of flood events. The 1986 flood eroded 30 percent of the tunnel lining thru to bedrock in the Lowell Creek tunnel. One large hole measured 125 feet long, 10 or more feet deep, and 25 feet wide. The original tunnel was 10 feet in diameter. The Fourth of July levee suffered considerable damage during the 1986 storm. The damage was partly due to scour at the toe of the levee which was underestimated during the original design. An empirical approach based on hydraulic mean depth and scour data from flume studies and gravel bed rivers in Alaska and Canada was used to estimate scour depths. This paper describes both flood control projects, problems that have occurred to date and presents a proposed feasibility level plan of study for the Lowell Creek project. The main objective of the project is to reduce the risk of loss of life and property damage to the downstream population.

DEPARTMENT OF THE ARMY
UNITED STATES ARMY MATERIEL COMMAND
ARMY RESEARCH OFFICE
P.O. Box 12211, Research Triangle Park, NC 27709-2211

REPLY TO ATTENTION OF:

AMXRO-RT-IP
WES/77
27401-GS

20 Aug 92

MEMORANDUM FOR Commander, Waterways Experiment Station, ATTN: CEWES-GV-Z (Monte L. Pearson), P. O. Box 631, Vicksburg, MI 39180-0631

SUBJECT: SL's Copy, Semiannual Status Report, ARO Proposal No. 27401-GS

1. The Army Research Office requires a semiannual report from its contractors. This report serves to briefly review important Army interactions, significant research progress, and technology transfer during the reporting period. It is not intended to give a comprehensive overview of the project. Because of your interest in this project, we are sending you a copy of the semiannual report.

2. SL representatives are encouraged to visit the project to obtain a comprehensive review of the work. Please note, however, that the principal investigator bears the responsibility of pursuing the research. The individual exercising SL must avoid any semblance of control, supervision, or redirection of the work. We are, of course, interested in receiving your comments on the project after a site visit, especially with regard to significant progress and potential for application.

3. We welcome comments from both SL and SC representatives on the enclosed report. Please address any comments to: Director, U.S. Army Research Office, ATTN: AMXRO-RT-IP, P.O. Box 12211, Research Triangle Park, NC 27709-2211.

FOR THE DIRECTOR:

Encl

/s/
GEORGE A. NEECE
Director
Research and Technology Integration

PROGRESS REPORT

TWENTY COPIES REQUIRED

1. ARO PROPOSAL NUMBER: 27401-GS

2. PERIOD COVERED BY REPORT: 1 January 1992 - 30 June 1992

3. TITLE OF PROPOSAL: Computer Simulation of Subaqueous Sediment Transport

4. CONTRACT OR GRANT NUMBER: DAAL03-89-K-0163

5. NAME OF INSTITUTION: Duke University

6. AUTHORS OF REPORT: Peter K. Haff

7. LIST OF MANUSCRIPTS SUBMITTED OR PUBLISHED UNDER ARO SPONSORSHIP DURING THIS REPORTING PERIOD, INCLUDING JOURNAL REFERENCES:
   "Transport of Granules by Wind and Water: Micromechanics to Macromechanics in Geology and Engineering, P.K. Haff et al., in Adv. in Micromechanics of Granular Materials, H.H. Shen, et al., eds., pp 373-380, Elsevier Sci. Pub., 1992.

8. SCIENTIFIC PERSONNEL SUPPORTED BY THIS PROJECT AND DEGREES AWARDED DURING THIS REPORTING PERIOD:

   P.K. Haff, Z. Jiang, T. Burnet

9. REPORT OF INVENTIONS (BY TITLE ONLY):

Peter K. Haff
Department of Civil & Environmental Engineering
Duke University
Durham, NC 27706

Brief Outline of Research Findings

We have continued our subaqueous simulation work in the areas of mixing of sediments, swash action on a beach, friction or pivot angles in three dimensions, and effects of imbrication of irregularly shaped particles on pivot angle stability. A general discussion of the simulation approach has been submitted to Water Resources Research, and the mixing and swash results, to be published later, are summarized in the recent thesis of Z. Jiang. T. Burnet is continuing to work on three-dimensional problems.

In the swash simulations, the multi-slab fluid model described in our last report has been applied to look at sediment transport on beaches of different slope and with different grain sizes. The infiltration of water into the beach has been shown to be an important process. At realistic infiltration rates, the backwash portion of the swash/backwash cycle is much weaker than when infiltration is absent. Consequently, upslope sediment transport increases rapidly with increasing infiltration rate. For a given infiltration rate, increasing beach slope tends to decrease onshore transport. At some critical slope, net upslope transport ceases. Because coarser-grained beaches generally have higher infiltration rates, they tend to become steeper than fine-grained beaches.

Studies of bed stability in three dimensions have been extended to show the variation in individual pivot angles as a function of direction of tilt. Two angles are necessary to specify stability in three dimensions. The individual particle results are averaged over the azimuthal tilt direction to get the average angle of bed stability for the simulated bed. The results are being compared to two-dimensional studies to assess quantitatively the influence of the third dimension. These results are also being compared to friction angles for non-spherical imbricated particles (in two dimensions). Imbricated beds are formed by allowing a simulated flow to "process" or "work" the bed prior to measurement of friction angles. In this way, fairly realistic sediment configurations can be studied and their differences with idealized (spherical) particle beds assessed.

"PARTICLE DYNAMICS AND GRAVEL STREAM-BED ADJUSTMENTS"

Fourth Interim Report

by

Dr P A Carling

August 1991

United States Army
European Office of the US Army
London, England

CONTRACT NUMBER # DAJA45-90-C-0006

Freshwater Biological Association

The Research reported in this document has been made possible through the support and sponsorship of the US Government through its European Research Office of the US Army. This report is intended only for the internal management use of the contractor and the US Government.

REPORT DOCUMENTATION PAGE

Contract Number: DAJA45-90-C-006

Report Number: FOUR Period Covered: from December 1990 to August 1991.

Name of Institution: Freshwater Biological Association.

Principle Investigator: Dr. P A Carling

Abstract:

The results of an highly successful field season are reported, together with brief details of subsequent preliminary data analysis and plans for the future.

REPORT

The following statements detail investigations conducted over the period January to August 1991.

During the winter months Prof. Ergenzinger developed a traversing echo-sounder for installation at Squaw Creek, and students under the direction of Prof. Christaller rebuilt the magnetic detector logging equipment. Dr. Carling investigated the use of acoustic pebbles for tracing studies and placed an order with a Canadian company for development work.

In April research team personnel re-equiped the field site at Squaw Creek and added a traversing mechanism to the upstream bridge for positioning of electromagnetic current-meters at intervals across the section. The new magnetic detector system was installed and improved during trials until excellent results were obtained: individual particles proving clear unambiguos signals. During late April through to early June high quality field data were collected.

A considerable snow-pack this year and high air temperatures meant that high run-off rates occurred, unlike 1990. Long continuous multi-day records of pebble transport were obtained using the detector log. Preliminary observation indicated that transport was in waves approximately every 30 secs. These records were supplimented by detailed hydraulic data collected during two large hydrographs over 24 hr periods. Data include, depth, velocity, and shear stress distributions as well as fluctuations in bed level which can be correlated with the bedload records. The traversing echo-sounder failed to work owing to cavitation problems around the head, but it is hoped to solve these problems for the 1992 season. Dr Carling still had not taken delivery of the acoustic pebbles but it is anticipated that these will be tested in the UK in time for next year.

Dr Jon Williams of the Bidston Observatory of the UK Institute Of Oceanographic Sciences visited the site following a request for collaboration from Dr. Carling. The latest generation of fine resolution electromagetic current meters were used to obtain high quality turbulence data and, in addition, signal-generated noise of mobile gravel was recorded, for later comparison with bedload transport rates obtained in a net-trap and with the detector log records.

A demonstration of field techniques and presentations using poster displays were provided for Monte Pearson (WES) and Jerry Comati (ERO) over a two day period in early June.

On return to the Europe in June, the detector records have been processes to remove any remaining noise or spurious spikes, the velocity data have been processes to provide summary data sheets and the turbulence data have been subject to preliminary examination.

Plans have been made for the prime movers to meet in Berlin in the fall to exchange data, plan the content of the final report and to discuss possible publication. The opportunity will also be taken to discuss the scientific programme for 1992 (the final year) and the future of the Squaw Creek site with respect to seeking additional sponsorship to maintain the facility and continue hydraulic investigations.

"PARTICLE DYNAMICS AND GRAVEL STREAM-BED ADJUSTMENTS"

Fifth Interim Report

by

Dr P A Carling

12 December 1991

United States Army
European Office of the US Army
London, England

CONTRACT NUMBER DAJA45-90-C-006

Freshwater Biological Association

The Research reported in this document has been made possible through the support and sponsorship of the US Government through its European Research Office of the US Army. This report is intended only for internal management use of the contractor and the US Government.

REPORT DOCUMENT PAGE

Contract Number:	DAJA45-90-C-0006
Report Number:	FIVE
Period Covered:	12 June 1991 to 12 December 1991.
Principle Investigator:	Dr. P A Carling

Abstract:

Data analyses completed and future plans are reported.

REPORT

The following statements detail investigations conducted over the period June to November 1991.

Dr. Carling has taken delivery of the acoustic pebble tracing system, but there has not been an opportunity to test these in the field. It is anticipated that trails will be carried-out over the winter, for development in the field season in Montana in the spring of 1992.

Dr. Jon Williams of the Bidston Observatory of the UK Institute Of Oceanographic Sciences completed his preliminary inspection of the turbulence data. Some high quality short data runs have been obtained which qualitatively demonstrate periodicities in phase with the passage of sediment waves. However, the nature of the turbulent spectrum in the mountain stream was not as anticipated and there is opportunity to improve the instrumentation. This is to be done over the winter and Dr Williams expects to deploy the turbulence rig in the spring of 1992.

Considerable correspondence and dialogue has taken place between Prof Steve Custer of Montana State University, Prof Peter Ergenzinger of the Free University of Berlin and Dr Paul Carling concerning the research programme in 1992. It has been concluded that accurate energy slope records are required with a fine temporal resolution. Prof. Custer is exploring funding sources for this instrumentation. This equipment is required because the bedload pulse data of 1991 exhibits quasi-regular periodicity, which appears to correlate both with turbulent pulses and fluctuations in the water surface. The latter factor is presently poorly measured using an array of stilling-wells.

Plans have been made for Dr. Carling and Prof Ergenzinger to meet in Berlin before Christmas to exchange data, plan the content of the final report and to discuss possible publication. The opportunity will also be taken to discuss the scientific programme for 1992 (the final year) and the future of the Squaw Creek site with respect to seeking additional sponsorship to maintain the facility and continue hydraulic investigations.

Prof. Ergenzinger presented a paper at a BGRG/COMTAG Symposium on "Theory In Geomorphology" which included aspects of the Squaw Creek project.

"PARTICLE DYNAMICS AND GRAVEL STREAM-BED ADJUSTMENTS"

Seventh Interim Report

by

Dr P A Carling

12 June 1992

United States Army
European Office of the US Army
London, England

CONTRACT NUMBER DAJA45-90-C-0006

Freshwater Biological Association

The Research reported in this document has been made possible through the support and sponsorship of the US Government through its European Research Office of the US Army. This report is intended only for internal management use of the contractor and the US Government.

REPORT DOCUMENT PAGE

Contract Number:	DAJA45-90-C-0006
Report Number:	SEVEN
Period Covered:	13 March 1992 to 12 June 1992
Principal Investigator:	Dr P A Carling

Abstract:

Data analyses completed and field investigations are reported.

REPORT

The following statements detail investigations conducted over the period March to June 1992.

The field data for 1991 have been analysed in a preliminary fashion. These data demonstrate bed level fluctuations during the passage of snow-melt hydrographs when bedload transport occurs. The passage of an unsteady wave of fine bedload over a coarser immobile bed leads to unexpected changes in hydraulic roughness as characterized by the Darcy-Weisbach roughness coefficient. This is contrary to the usual assumption that total bed roughness falls with rising discharge. This result has implications for modelling flood flows and sediment routing and will be investigated further.

As proposed in Report Five, four acoustic water level sensors were installed in the field in April 1992, to monitor water level fluctuations at 1 sec intervals during the passage of hydrographs. These proved an improvement over older float operated water level sensors.

A vortex-type bedload trap was constructed at the site and tested for sampling efficiency. Unfortunately, runoff this year was slight owing in adequate snow-pack and consequently no bedload transport occurred.

Dr Jon Williams of the Bidston Observatory of the UK Institute of Oceanographic Sciences completed a series of measurements of turbulent fluctuations in the flow. Preliminary indications are that these are of high quality and represent the first successful measurements of high-resolution fluctuations of current vectors in a mountain stream. In addition a hydrophone was calibrated to record only the passage of bedload transport, but could not be used in earnest owing to the low flows encountered. Additional measurements of the vertical velocity distribution over rough gravel beds were made using a small dorsally-flattened electro-magnetic current meter at one-half to one centimetre intervals in the vertical. These data have yet to be processed, but preliminary plots indicate high-quality definition of the structure and extent of the boundary-layer has been obtained.

Additional measurements were also made across the section using impellor flow-meters and electro-magnetic meters to determine the nature of secondary-flow circulation patterns.

Further improvement and calibration of the magnetic bedload detection system was undertaken by Prof Georg Christaller from Berlin.

Prof Ergenzinger presented a paper concerning results from 1991 at a Conference in Bavaria in June 1992.

RIVER DYNAMICS GROUP INTERIM REPORT

Guidebook of applied Fluvial Geomorphology for River Studies in engineering and Environmental Management

from

Department of Geography,
University of Nottingham,
Nottingham NG7 2RD

to

US Army Research, Development & Standardisation Group - UK

under project DAJA45-91-M-0172

Meetings

The River Dynamics Group are working on the Guidebook through a management structure consisting of chapter editors and section leaders. This structure was agreed by the group at the first meeting under this project, which took place at Nottingham in June 1991. Subsequent meetings have taken place in the course of professional conferences such as those at Vienna, August 1991 (IUGG), Leeds, September, 1991 (BGRG) and Swansea, January, 1992(IBG). The main item for discussion has been the contents and chapter break-down of the proposed *Guidebook of Applied Fluvial Geomorphology for River Engineering and Management.*

There was a very full discussion of the outline developed three years ago, especially in the light of suggestions put forward by Dr Keith Richards of the University of Cambridge. Dr Richards could not attend the meeting, but did send a long and detailed written contribution.

The RDG reviewed the basis for the book, its intended market and its relevance to the 1990s. After a long debate, it was decided to maintain the focus on geomorphology and to try produce a volume of use to engineers, fisheries & conservation officers and planners - more or less in that order. The group felt that in the future the guidebook could make an excellent text for short courses put on by the RDG for professionals working for action agencies such as the US Army Corps of Engineers, the National Rivers Authority and consultancy companies with interests in rivers and river environments.

Regarding the final format of the book, any decisions now would probably be premature. All that is required of us by the funded contract is a loose bound report (in multiple copies). Copyright remains with the writers. Quite how we present the manuscript is entirely up to the individuals who are writing the sections and will be addressed at subsequent RDG meetings. Therefore, nothing should be read into the groups' decision to use the authors' notes developed by John Wiley & Sons Ltd. as the guide to preparing the manuscript. This format was selected because it is already familiar to most of the group who publish in Wiley books and other Wiley journals and research monographs.

Following discussion with RDG members, Dr Phil Ashworth of the University of Leeds has been invited to join the group. He has indicated that he is delighted to do so.

The Guidebook

The length of the guidebook should be limited to about 500 pages (400 text + 100 diagrams etc.) and the content should be experience based. Each of the four main chapters should be of about 20,000 words (10,000 for introductory and closing chapters), and sections will 5,000 to 10,000 words in length.

The US Army-ERO has set no limits on the scope or content of the book, but the group felt that the case studies used should come from the UK and that the scope should be limited to mid-latitude, humid environments. It was recognised that the RDG has the expertise to cover much more of the world and that maybe we should address that - in the next project. However, there would be nothing wrong in pointing out how methods, techniques and analyses presented in the context of humid, mid-latitude rivers could be extended or applied to rivers in say, humid-tropical or semi-arid environments.

The main discussion then centred on the best mechanism for producing the manuscript, especially bearing in mind the "false starts" of the past. The group decided to review the outline contents of the book chapter by chapter, assigning responsibility to individuals for editing particular chapters. Within each chapter, major sections are to be written by small teams, led by a section coordinator. It is the responsibility of each section coordinator to ensure that the section progresses satisfactorily. The revised deadline for production of the manuscript is in summary:

Initial draft sections + chapters	June 1992
Final draft sections + chapters	October 1992
Complete manuscript	December 1992

The group then considered which members of the RDG should be asked to act as chapter editors, section coordinators and section writers. We also reconsidered the draft outline of sections and made some fairly minor changes. The revised outline, with the suggested editors, coordinators and writers, is attached. The plan is for those named in the revised outline to produce draft manuscripts, initially for review by the chapter editors and later for review by the whole group. Individuals who have papers, pre-prints, unpublished reports and other material which they wish to see referenced in particular sections should send these directly to the relevant section coordinator. Clearly, there is the potential for significant overlap between some sections. This should be handled firstly by direct contact between section coordinators during the writing of the sections and, secondly, when the draft sections are reviewed by the group.

The next full RDG meeting will be in on March 20, 1992 at Nottingham.

As you know, the RDG is an informal group with no officers, chair people, or hierarchy. While Richard Hey, Malcolm Newson and I took the initiative on obtaining funding for the production of this volume, and are willing to act as editors for the whole book, all the decisions made at the meetings to date were by common consent, and requests to individuals not present at the meeting to act as editors etc. are just that: *requests*.

If you have urgent comments or criticisms, or particularly wish to be (or not to be) an editor, coordinator or writer, please let me know a.s.a.p. I shall be in the USA on fieldwork from July 7 to 20 inclusive, but I will give any re-shuffling of responsibility a high priority when I get back.

REVISED AND UPDATED DRAFT CONTENTS FOR GUIDEBOOK OF APPLIED FLUVIAL GEOMORPHOLOGY FOR RIVER ENGINEERING AND MANAGEMENT

Chapter editors' names follow Chapter Title
Section writers' names are listed after each section
Section coordinators' names are underlined

Chapter 1. Introduction
(chapter editors: Richard Hey, Malcolm Newson, Colin Thorne)

Review of practical problems
Natural design approaches
Timescales in engineering and management contexts — <u>Hey</u>, Newson, Thorne

Chapter 2. Natural channel stability and time perspective
(John Lewin, Mark Macklin)

2.1 Long-term changes in channel stability
Effect of climatic change and
Quaternary sea-level changes
Human impacts on the environment
Basin sediment systems
Fluvial landforms in alluvial valleys — Lewin, <u>Macklin</u>

2.2 Intermediate-term changes in channel stability
Available sources of information
Rates of change in time
Basin scale sediment budgets
Erosion, transport and deposition of sediment
Regime channels and dominant discharge — Harvey, Hooke, Lewin, <u>Macklin</u>

2.3 Short-term changes in channel stability
Discharge and sediment transport regime
Sediment sources, supply limitations and response
Effect of rare, large magnitude floods
Triggering of geomorphic thresholds

Ferguson
Leeks
Werrity

Chapter 3. River Channel and Valley Processes
(James Bathurst)

3.1 Environmental river flow hydraulics
Characteristics of river flow
Secondary currents and shear stress distributions
Flow resistance of bed and bar roughness
Vegetation Effects
Field studies and techniques
Formulae, applications and limits to applicability

Bathurst
Best
Carling
Hey
Thorne

3.2 Sediment Erosion, Transport and Deposition
Characteristics in different channel types
Erosion and Deposition
Pulsing of sediment transport
Field studies and techniques
Formulae, applications, limits to applicability
Sediment associated pollutants

Bathurst
Carling
Frostick
Lewin
Macklin
Reid
Walling
Webb

3.3 Bank erosion, stability and retreat
Characteristics of natural banks
Field studies and techniques
Bank stability calculations
Bank retreat predictions

Hooke
<u>Lawler</u>
Thorne

Chapter 4. Channel morphology and design procedures
(Keith Richards)

4.1 Channel types and morphogenetic classification
Straight/meandering/braided
Sinuosity
Active and passive meandering
Confined and unconfined channels
Relation between channel pattern and x-section
Stream power controls on pattern and shape

<u>Ferguson</u>
Gregory
Hooke
Werrity

4.2 Styles of channel change
Modes and patterns of migration and evolution

Ferguson
<u>Hooke</u>
Werrity

4.3 Rational and regime techniques
Dominant and design discharge
Design of straight and meandering channels
using geomorphological approach

Brookes
<u>Hey</u>
Lewin
Richards

4.4 Prediction of morphological changes
Modelling erosion, deposition and channel change
in dynamic systems

Hey
<u>Richards</u>

Thorne

4.5 River dynamics and channel maintenance

<u>Brookes</u>
Newson
Thorne

Chapter 5. Case Studies*
(Malcolm Newson)

5.1 River stabilisation and channel design

5.2 Flood alleviation schemes

5.3 River regulation schemes
Reservoir operation, groundwater development, inter-basin transfers

5.4 Bank protection and stabilisation projects
Revetments, grade control, Iowa vanes, flow control

5.4 River restoration and management for environmental and habitat improvement

5.5 Design of channel crossings
Bridges, pipelines etc.

Brookes
Carling
Gregory
Hey
Leeks
Newson
Petts
Thorne
+ others*

* This chapter is different to the others and requires special handling. Please contact the chapter editor if you have a case study to contribute to this chapter in any of the above sections.

Chapter 6 Practical Approach to Using the Guidebook
(Richard Hey, Malcolm Newson, Colin Thorne)

LIST OF CONTRIBUTORS

Name	Title	Department	Institution
Ashworth, Phil	Lecturer	Dept Geography	Univ. Leeds
Bathurst, James	Principal Scientist	NERC Rivers Unit	Univ. Newcastle
Best, Jim	Lecturer	Dept. Earth Science	Univ. Leeds
Brookes, Andrew	Geomorphologist	River appraisals	Thames NRA*
Carling, Paul	Principal Scientist	Fisheries	NERC, IFE**
Ferguson, Rob	Professor	Dept. Geography	Univ. Sheffield
Frostick, Lynne	Senior Lecturer	Graduate Inst.	Univ. Reading
Gregory, Ken	Professor	Dept. Geography	Univ.Southampton
Harvey, Adrian	Senior Lecturer	Dept. Geography	Univ. Liverpool
Hey, Richard	Senior Lecturer	Scl.,Env. Science	Univ. East Anglia
Hooke, Janet	Senior Lecturer	Dept. Geography	Portsmouth Poly.
Lawler, Damian	Lecturer	Schl. Geography	Univ. Birmingham
Leeks, Graham	Senior Scientist	Plynlimon Site	NERC, IH***
Lewin, John	Professor	Dept. Geography	Univ. Wales
Macklin, Mark	Sen.Res. Assoc.	Dept. Geography	Univ. Newcastle
Newson, Malcolm	Professor	Dept. Geography	Univ. Newcastle
Petts, Geof	Professor	Dept. Geography	Univ Loughboro'
Reid, Ian	Reader	Dept. Geography	Birkbeck College
Richards, Keith	Lecturer	Dept. Geography	Univ. Cambridge
Thorne, Colin	Professor	Dept. Geography	Univ. Nottingham
Walling, Des	Professor	Dept. Geography	Univ. Exeter
Webb, Bruce	Senior Lecturer	Dept. Geography	Univ. Exeter
Werrity, Alan	Senior Lecturer	Dept. Geography	Univ. St Andrews

* National Rivers Authority
** Institute of Freshwater Ecology
*** Institute of Hydrology

REPORT DOCUMENTATION PAGE	Form Approved OMB No. 0704-0188

1. AGENCY USE ONLY *(Leave blank)*	2. REPORT DATE September 1994	3. REPORT TYPE AND DATES COVERED Final report

4. TITLE AND SUBTITLE
Streams Above the Line: Channel Morphology and Flood Control; Proceedings of the Corps of Engineers Workshop on Steep Streams

5. FUNDING NUMBERS
Work Unit 32553

6. AUTHOR(S)

7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES)

8. PERFORMING ORGANIZATION REPORT NUMBER
Miscellaneous Paper HL-94-4

9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES)
U.S. Army Corps of Engineers
Washington, DC 20314-1000

10. SPONSORING/MONITORING AGENCY REPORT NUMBER

11. SUPPLEMENTARY NOTES
Available from the National Technical Information Service, 5285 Port Royal Road, Springfield, VA 22161.

12a. DISTRIBUTION/AVAILABILITY STATEMENT
Approved for public release; distribution is unlimited.

12b. DISTRIBUTION CODE

13. ABSTRACT *(Maximum 200 words)*

The U.S. Army Corps of Engineers Workshop on Steep Streams was held in Seattle, WA, 27-29 October 1992, to provide an opportunity for presentation by Government, academia, and private industry on research in steep streams/gravel rivers/high-velocity flow. Sixteen papers presented at the meeting are included in this report.

14. SUBJECT TERMS
Gravel-bed streams
High-velocity flow
Steep streams

15. NUMBER OF PAGES
322

16. PRICE CODE

17. SECURITY CLASSIFICATION OF REPORT	18. SECURITY CLASSIFICATION OF THIS PAGE	19. SECURITY CLASSIFICATION OF ABSTRACT	20. LIMITATION OF ABSTRACT
UNCLASSIFIED	UNCLASSIFIED		